AN INVESTIGATION OF THE SWELLING MECHANISM OF YAZOO CLAY

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AN INVESTIGATION OF THE SWELLING

MECHANISM OF YAZOO CLAY

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DEDICATION

This thesis is dedicated to my wife for her patience and encouragement over the past several years.

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SUMMARY

The Yazoo Clay is one of many formations found in this country and in several foreign countries which, due to their physical and chemical properties, undergo large volume changes when subjected to changes in moisture content. This property results in difficult foundation problems which have become more accute in recent years, especially with light structures. The increase in construction utilizing materials susceptible to movements focussed much attention on this problem in those areas where expansive soils exist.

In general, this investigation deals with the mechanism of swelling of expansive clay soils. In particular, it is concerned with characteristics of the Yazoo Clay. The Yazoo Clay was chosen for this study because it is believed to be typical of expansive clays and is one of the few formations of such materials which occur in a heavily populated region of the Southeastern United States.

Most early work on swelling mechanisms was confined to gels. In 1925 Terzaghi published a paper in which the similarity between the swelling of ealstic gels and the expansion of coarse grain soils was discussed. Since then, there were few publications dealing with the swelling of soils, from an engineering standpoint, until about 1950. Over the past decade research into the problem of swelling soils has increased rapidly, most work dealing with swell pressures developed and the determination of swelling potential. Minor work has been done

on prediction of heave under field conditions. The majority of this research was done on remolded or laboratory prepared specimens.

This investigation was undertaken to study the swelling and shrinkage mechanism of undisturbed specimens of the Yazoo Clay. The study is particularly concerned with moisture content-swell pressure relations, vertical and lateral swell pressures, the effect of preferred orientation on shrinkage and swelling, and the effect of several salts on swelling.

The Yazoo Clay is a highly calcareous marine deposit ranging in thickness from 100 to 500 feet which covers a narrow band across central Mississippi. Undisturbed specimens were obtained in four-inch diameter thin-walled tubes in an open pit at a depth of approximately twenty feet from the original surface. All of the specimens were obtained from as small an area as practical at approximately the same depth.

To obtain the desired objectives a broad testing program including swell pressure tests measuring both vertical and lateral pressures was conducted. For these tests a special device was constructed utilizing glycerin as a lateral confining fluid. The lateral pressure was measured by means of an electronic transducer placed in contact with the glycerin; vertical swell pressure was transmitted through a piston to a restraining yoke instrumented with SR-4 electric strain gages.

Other tests were conducted in 1.40 inch diameter and 2.40 inch diameter consolidometers. Standard index properties were determined for identification and correlation.

Sixteen tests were conducted in the swell pressure cell; the only deformation allowed was that which results as a characteristic of the cell. The data were divided into two groups based on the liquid limit and plasticity index. Group One, with an average liquid limit of 65 and plasticity index of 35, had a linear relation between initial moisture content and swell pressure. The data for Group Two, having an average liquid limit of 69 and a plasticity index of 41, were non-linear with the maximum difference from the data of Group One occurring near the natural shrinkage limit. At this moisture content the curve for Group Two indicates a swell pressure of almost twice that of Group One. The problems involved in predicting swell pressures in natural soils due to variation of soil properties are clearly demonstrated by these data. For moisture contents of from 1.1 to 1.2 times the plastic limit the swelling is negligible.

A plot of the swell pressure-void ratio data as compared to consolidation test data indicates that the pressure required to consolidate a soil to a given void ratio is much greater than that which will be produced by swelling of the same soil when restrained at the same void ratio and allowed to freely imbibe water.

A comparison of the vertical and lateral swell pressures indicates that the swell mechanism is of a hydrostatic nature. For ratios of vertical to lateral strain of less than two, the vertical and lateral swell pressures are approximately equal. For values greater than two the soil is no longer able to adapt itself to the confining conditions and the larger allowed deformations result in lower swell pressures.

A plot of the moisture content-void ratio data indicates that shrinkage of the Yazoo Clay has a linear relation with moisture content change, until the natural shrinkage limit is reached. In this range, the change in the degree of saturation is small. Below the natural shrinkage limit the resistance to shrinkage is greatly increased, resulting in a small change in void ratio from the natural shrinkage limit to an oven-dried condition. The natural shrinkage limit is much higher than the shrinkage limit as determined by standard methods. These data indicate that the shrinkage limit as determined by standard methods should not be used as a lower limit of shrinkage capabilities of an undisturbed natural soil.

The linear shrinkage data indicate that the vertical shrinkage of the soil tested from near the plastic limit to an air-dried condition is 1.95 times the lateral shrinkage. For remolded specimens the ratio is 1.13. This difference in vertical and lateral shrinkage is attributed to a preferred orientation of the clay particles and emphasizes the anistropic nature of the soil.

To study the effect of anistropy on swelling, tests were conducted on specimens loaded normal and parallel to the bedding plane of the soil. The results of these tests indicate that the swell normal to the bedding plane is about 2.3 times that obtained parallel to the bedding plane. It was also noted that less consolidation occurred for loads applied parallel to the bedding plane than for loads applied perpendicular. There was some reorientation of particles in those specimens loaded parallel to the bedding plane. Based on shrinkage measurements, it appears that the particles have a tendency to orient themselves perpendicular to the direction of loading. This is in agreement with published data.

In conducting the swell pressure tests, it was noted that some of the specimens continued to swell at a very slow rate after the initial swelling had been completed. Tests of 43 and 94 days were still showing a slight but steady increase of pressure with time when terminated. This slow rate of swelling may possibly be attributed to a long time hydration of the adsorbed ions near the clay mineral surface because of the extremely high forces occurring between the clay mineral, water molecules and adsorbed ions.

To explain the unusual deformation-swell pressure relations obtained, an idealized soil with all particles the same size, oriented at a 2:1 slope with a horizontal plane, is used. With the edge to face bonding of particles, swelling is assumed to result in the particle edges being pushed outward from the face. Under those conditions it is shown that a verticle movement can be obtained equal to four times the lateral. The vertical and lateral forces required to prevent such swelling are equal. This idealized soil also indicates that only a slight orientation of the particles is necessary to obtain large ratios of vertical to lateral shrinkage.

Results of double oedometer tests on the Yazoo Clay indicate that this test is not valid, at least to the extent of adjusting the e-log p curves so that the virgin portions coincide.

Tests were conducted to study the effect of calcium and sodium chloride on swelling by submerging undisturbed specimens in several concentrations of the salts. Based on the calcareous nature of the Yazoo Clay, the adsorbed cations are assumed to be calcium at a concentration of about 2.5 molar. With increased calcium chloride concentrations the magnitude of swelling was decreased. At approximately five molar, the decreasing rate of swell is diminished and the magnitude is about one-half of the swell obtained for distilled water. There was no noticeable change in the data when the pore water ion concentration was changed from below to above the calculated concentration of adsorbed cations.

For the specimens submerged in sodium chloride, an increase in swelling was noted for concentrations of two molar as compared to distilled water. Above this value the swelling decreased. For concentrations of 5.3 molar the swelling was approximately 0.8 of that for distilled water.

The decrease in swelling produced by the calcium chloride is attributed to a reduction of osmotic pressure between the pore water and adsorbed water for concentrations up to those of the adsorbed water. For values above this, the further reduction may be attributed to a shielding effect of the high cation concentration resulting in a reduction of electrostatic repulsion between clay particles.

For the sodium chloride tests, it is concluded that an exchange of sodium ions for calcium ions occurs resulting in the increased swelling. At the higher concentrations, the reduced swelling is attributed to the nonexistence of an osmotic condition after the exchange has occurred, i.e. the concentration in the pore water is equal that of the adsorbed layer. It is concluded that for the soil tested:

(a) The swell pressure developed at near zero volume change is proportional to the initial moisture content.

(b) The soil will not swell when the moisture content is about 1.2 times the plastic limit.

(c) Long-time swelling occurs which is not detected under usual test conditions; it can become significant.

(d) The swell pressure-void ratio relation is not the same as the consolidation pressure void-ratio relation.

(e) The shrinkage characteristics of a soil may be used to indicate nonisotropic conditions.

CHAPTER I

INTRODUCTION

Significance of the Problem

The Yazoo Clay is one of many clay formations found in several areas of the United States and in many foreign countries which, due to their physical and chemical properties, undergo significant changes in volume when subjected to a change in moisture content. The change in volume of such clays may appear as vertical movements of the ground surface, the opening and closing of cracks in the soil or, under certain conditions, horizontal movements. Obviously, such movements can be damaging to any structure resting on such a clay. The changes in moisture content necessary to produce shrinking or swelling of a clay soil may result from seasonal climatic conditions or environmental changes within or on the ground surface.

Such soils create very difficult and unusual foundation problems. In general, the difficulties involved in foundation design increase as the size of the structure increases. With the increased value of the structure, more money can be spent on the foundation. However, in expansive soils, the most difficult problems occur with light structures, the value of which prohibits extensive foundation costs. The problem is by no means confined to light structures, but it is in this area where the largest volume of damage occurs.

Unfortunately, failures in light commercial buildings and residences are not generally spectacular and go unnoticed, except by the owner. Many such structures are not engineered, just built; the net result has been a lack of interest and, hence, a lack of knowledge of the problems.

This lack of interest has been somewhat overcome in the past ten or fifteen years. A study of the literature on the problem would almost lead one to believe that the problem did not exist or was of academic interest only prior to this time. Although expansive clay formations have always been with us, the related foundation problems have not always been as acute. In the past few years, this problem has become more significant because of changes in type of construction, building materials, more stringent building regulations, magnification of the problem because of expanded construction and an increased interest in the field of soil mechanics and foundation engineering. There are undoubtedly many other factors involved.

One of the interesting aspects of this problem is the effect on the public. Probably no other foundation problem has become the concern of so many individuals outside of the engineering field. The seemingly mysterious movements of residences has been the subject of much concern, conversation and speculation.

In view of the seriousness of the problem, it is surprising that so little is known about expansive clays, even by practicing engineers. The author has had contact with two projects in recent years in which structures were underpinned because it was believed that portions of these buildings were settling due to consolidation of the foundation soil. In both cases, investigations showed that movements were due to shrinkage or swelling of an underlying expansive clay.

The Yazoo Clay was chosen for this investigation because it is representative of the overall problem and because it is one of the few expansive clays in the Southeastern United States which occurs in a fairly heavily populated area. In addition, to this author's knowledge, there has been no extensive research on the swelling mechanism of this material.

Investigation of related literature indicates that very little swell testing has been conducted on undisturbed soil samples. Most of the work which has been reported utilized laboratory manufactured soils, i.e., mixtures of pure sands and clay minerals, or remolded natural soils. Such specimens result in more consistent test results. To obtain basic knowledge and to study compacted soils, this approach is essential; but it is also necessary that undisturbed specimens be tested if this knowledge is to be extended to field problems in which both isotropic and non-isotropic soils are encountered. It is realized that a testing program using natural undisturbed specimens presents many problems in the control of variables and in the interpretation of data.

In general, this investigation deals with the mechanism of swelling of expansive clay soils. In particular, it is concerned with these characteristics of the Yazoo Clay.

Historical Review

The study of the swelling and swelling pressures of gels and colloids dates back to at least 1885 (28). In 1912 relationships between moisture concentration and pressure for gels were obtained by Posnjak (28), and in 1921 Bartell and Sims (2) published a paper pertaining to the swelling of colloidal material and containing a review of seven theories on swelling that had been offered prior to that time.

Possibly the first link between soil mechanics and the study of swelling of gels came in 1925 (39) and in 1931 (38) in papers presented by Karl Terzaghi. In these papers both elastic rebound and osmotic pressure are discussed. Both of these factors are still considered as important components of swelling.

The early work in soil mechanics was hampered by the inability of researchers to determine with any degree of confidence the structure of clay minerals and their orientation in soils. With the development of X-ray defraction equipment and the necessary techniques, it became possible to determine the structure and to properly identify the various clay minerals.

The development of the electron microscope made possible the determination of the true shape of clay particles and their orientation in natural and remolded soils. The work of Rosenquist (32) in this field is of notable interest.

Paralleling the early work on expanding gels, considerable research was being done in both agriculture and ceramics dealing with ion exchange. The effect of adsorbed cations on the properties of clay soil, especially from the standpoint of swelling and shrinking, has proven to be an important factor. The work of Winterkorn (43), Hauser (14) and Kelly (20) is significant in this respect.

In the field of clay mineralogy, the contributions of Grim (13) and others have added much to our knowledge of the physical and chemical properties of the clay minerals.

Research on expansive clay soils has, for the most part, been done since 1950; this will be reviewed in the next chapter.

Although the study of foundations on expansive clays is an engineering problem, it should be evident from the above brief summary that it is a problem involving chemistry, physics and mineralogy.

CHAPTER II

REVIEW OF LITERATURE

Introduction

The mechanism involved in the swelling of clay has been shown to be closely associated with the soil structure. Early investigations were hampered by the lack of knowledge pertaining to the nature of clay particle shapes and arrangement. It was 1923 before X-ray methods proved that clays were crystalline and not amorphous. With the determination of the structure of clay particles, several theories as to the nature of the structure of clay masses developed. Only very recently, with the developments in electron microscopy, did it become possible to determine with a high degree of confidence a true picture of clay structure (32). Oddly enough this work has shown that the particle arrangement of marine clays closely corresponds to the structure proposed by Goldschmidt in 1926 (32).

Many foundation engineers now believe that the structure as presented by Lambe (22) represents the correct arrangement of soil particles. For a complete discussion of clay structure and the interparticle forces involved reference should be made to this work. As described by Lambe, the cohesive fraction of a soil consists of plate shaped particles surrounded by a layer of adsorbed water and oriented in various arrangements from completely random to a high degree of parallelism. The orientation depends on sedimentation (salt water or fresh water) and previous stress history.

The soil exhibits strength due to the presence of attractive forces acting between clay particles and between the surface of the clay particles and water molecules. Repulsive forces also act between the particles. The degree to which a soil is consolidated determines the state of equilibrium between the attractive, repulsive and external forces in a soil mass.

Swelling is produced when the equilibrium is disturbed so that the repulsive forces become greater than those which are trying to force the particles together. Speculation as to the nature of the phenomena which unbalances this equilibrium has led to several theories of swelling.

Theories of Swelling

All of the early efforts to study swelling phenomena of colloids were confined to elastic gels. As early as 1885 it was observed that a volume change-pressure relation existed for gels imbibing a liquid. In 1912 Posnjak (28) reported pressure-volume change data for a gel obtained by placing the gel in a pressure cell and balancing the movement of confining mercury through a connecting tube with compressed gas.

In 1925 Terzaghi (39) attempted to explain the similarity which he noted between the expansion of coarse-grained soils and the swelling of elastic gels. In this paper the relation of swelling pressure and vapor pressures as derived by J. R. Katz is compared to the empirical relations obtained by H. Freundlich and E. Posnjak from many oedometer tests (swell pressure tests conducted in a consolidometer).

Terzaghi reported that the values given by the Katz's equation,

$$P = -\frac{RT}{MV_o} \ln h \tag{1}$$

where M = the molecular weight of the liquid contained in the gel,

V_o = the specific volume of this liquid,

T = the absolute temperature,

R = the gas constant,

P = the pressure exerted by the gel at a certain concentration per unit of surface of rigid obstacle with swelling and

h = the relative vapor pressure of the gel at the same concentration,

and those given by the empirical relation of Freundlich and Posnjak,

$$Q = Q_o\ C^k \tag{2}$$

where Q = swelling pressure in grams per cm^2,

C = grams of dry matter per 1000 cc of dry matter and liquid,

k = a coefficient and

Q_o = a content, characteristic for the gel

indicate a definite discrepancy.

He uses this discrepancy between the theoretical and empirical results to point out that the lowering of the vapor pressure may be due not only to mechanical stresses in the pore water, but possibly to "an alteration of the molecular state of the water," (39) p. 63. It is also shown by Terzaghi that the surface tension of water in the pores of gels must be considerably greater than the normal value. The similarity between the Katz equation and the formula,

$$P = \frac{RT}{MV_o} \ln \frac{h}{h_1}$$

where R = the gas constant,

T = the absolute temperature,

M = the molecular weight,

V_o = the specific volume of the solution,

P = osmotic pressure and

h and h_1 = the vapor pressure of the pure liquid and of the solution respectively,

for osmotic pressure is noted with the following comment (39), P. 78:

> If P in Equation (1) were identical with the swelling pressure measured with the oedometer, osmotic pressure and swelling pressure should be identical. It is surprising to notice that the identity of these formulas was known; the essential difference which exists between the osmotic pressure and the swelling pressure was also known. Nevertheless, it was believed that P in Formula (1) represented the swelling pressure.

Terzaghi left no doubt that he did not think that swelling pressure and osmotic pressure were the same. Unfortunately, he did not elaborate on this point. In an earlier paper, Bartell and Sims, 1921 (1), presented and gave a brief discussion of five theories of swelling of gels which had been put forth prior to that time. Osmotic swelling was one of those cited. These theories pertain generally to gels and are very limited in their scope so that a more detailed discussion is not warrented.

According to Terzaghi, the swelling pressure measured by the oedometer test is due to the elastic expansion of the solid phase which is released when the negative stress in the pore water is reduced to zero by immersing the gel in water. His final conclusion states that the pressure measured by the oedometer method is independent of the adsorption phenomena.

In a later paper, Terzaghi, 1931 (38), pursues further the capillary pressure-swelling process. To illustrate this process, a simple mechanistic system is used in which two identical saturated soil specimens were subjected to a decrease in height, one by an external load and the second by evaporation. At the identical reduced void ratio, the externally applied load is equal to the tension in the pore water of the desiccated specimen. If the external load is removed with the specimen submerged, an instantaneous tensile stress occurs in the pore water which is dissipated with time as water flows into the soil due to the difference in gradients. Swelling occurs due to mechanical energy of the compressed system. It is concluded that the swelling pressure (pressure required to prevent swelling) is equal to the pressure required to compress the soil to a given water content and the tensile stress in the pore water when the soil is consolidated to the same void ratio by desiccation. Again Terzaghi (38) concludes that the swelling pressure is the result of releasing inherent mechanical energy and that the physico-chemical energy has no influence on swelling pressure.

Terzaghi and Peck (40), p. 15, later described the cause of swelling as follows:

> If the water content of a soil sample decreases on application of pressure, the soil is said to consolidate. If at any stage the pressure is removed while the soil remains in contact with free water, the water content and volume increase. This phenomenon

> is known as "swelling". Each of the various grain-size fractions of a given soil exhibits the property of swelling, but to a different degree. The causes of the swelling are likely to be different for different fractions. If the pressure is removed from a coarse-grain sand-mica mixture, the mixture swells to a large degree, but the swelling is caused only by elastic restitution of the grains. If the pressure is removed from the very fine fraction of a clay, one part of the swelling is due to elastic restitution, and another part is likely to be due to an increase in the thickness of the adsorbed layers which separate the grains. In clays with a large percentage of montmorillonite, a third though very small part of the swelling is caused by the swelling of the montmorillonite particles themselves.

Power, Towle and Plaza (29) conducted swell pressure tests on specimens of Wyoming bentonite, shale from a depth of 8,470 feet and other "as received" soils. The bentonite was electrodialyzed and treated to provide predetermined exchangeable bases. The other soils were oven dried (105° C) and pulverized only. The test cell was similar to that of Posnjak with the pulverized soil being tamped and then compressed under static load into a very dense specimen. The static load used in compressing the specimen was determined from an experimental curve of pressure versus density. The tests were conducted under controlled temperature conditions with temperature and density corresponding to those encountered at depths of several thousand feet. The specimens were held at near constant volume and allowed to imbibe either distilled or chemically treated water. With distilled water, pressures in the neighborhood of 4800 psi were obtained in the bentonite. The shales produced pressures of approximately 1700 psi.

The authors attributed the swelling to "hydration" of the soil. What this term implies was not discussed. This is especially interesting in view of the authors' citing of Hauser (14) as stating that the thickness of water films encountered in clay systems can not be accounted for by the hydration of counter-ions. Actually, Hauser was referring to a clay system of much higher moisture content than those tested by Power, Towle and Plaza (29).

The effects of the various adsorbed ions are shown, but no attempt was made to analyze the data.

Hemwall and Low, 1955 (15), obtained pressure-volume and pressure-unfrozen water relations for sodium, thorium and a combination of sodium and thorium bentonite gels made from Wyoming Bentonite. The treated clay was placed in a pressure cell and allowed to swell against a predetermined pressure and then compressed by increasing the pressure in predetermined increments.

Ions of high valence are generally believed to disassociate much less than monovalent ions (14). In addition, fewer polyvalent than monovalent ions will be adsorbed on the mineral surface. By comparing sodium with a valency of one and thorium with a valency of four, the effect of disassociation and the number of osmotically active adsorbed ions could be observed.

As would be expected, the thorium treated clay swelled much less than the sodium clay. From X-ray evidence obtained on the swelled specimens, the authors conclude that the swelling of the thorium clay was interparticle and not interlayer. Interparticle swelling results from the clay particles being pushed apart; whereas, interlayer swelling results from expansion of the layered clay particle. It is further concluded that interparticle swelling occurs only at low pressures and is of small magnitude in relation to interlayer swelling.

In an effort to separate some of the possible causes of swelling, Hemwall and Low (15) attempted to eliminate van der Waals' forces (secondary bonds resulting from electrical dipoles) between the clay and the water. The treatment also eliminated the disassociation of cations from the external mineral surfaces. Swelling of the treated thorium clay was reduced for pressures less than three atmospheres; above this, the pressure-volume curves coincide. This evidence is used to substantiate the conclusion that the interparticle swelling is of a small magnitude and occurs only at low pressures.

Dioxone, a nonpolar solvent, which is very effective in breaking up hydrogen bond, was used by Hemwall and Low (15) to treat sodium clay to study the effect of hydrogen bonding on the swell characteristics. For the concentrations used, it was found that hydrogen bonding between clay and water is not of sufficient extent to effect clay swelling.

Bolt, 1956 (4), in discussing the compressibility of pure clays, i.e. soils containing only clay size particles, regards a pure clay as an osmometer where the semi-permeable membrane is formed by the clay particles. He describes the mechanism in which, by applying a load to the soil system, pore liquid is forced out until the osmotic pressure between the adsorbed liquid and the free liquid phase equals the applied loading pressure. According to Bolt, the ionic concentration close to the clay plates is between 2.5 and 5.0 molar which, if the plates become close, may result in osmotic pressures of between 50 to 100 atmospheres.

One important difference between highly compressible coarse-grained soil and pure clays pointed out by Bolt is the ratio between swelling index (slope of the rebound curve for consolidation test data plotted on semilogarithmic paper) and the compression index (slope of the virgin part of the compression curve for consolidation test data plotted on semilogarithmic paper). This ratio for highly compressible coarse-grained soil can be as low as 0.1; whereas, for pure clay the ratio is close to unity. He concludes that this ratio should in a general way be indicative of the relative importance of osmotic forces in the compression of clay soils.

In comparing the mechanical and osmotic concept of swelling, Bolt (4), p. 88, states:

> Thus the swelling pressure of the system is caused by the tendency of the liquid phase to re-enter the system. The mechanical concept mentioned before gives a similar explanation of the swelling of flaky materials, except that in this theory the tendency of the liquid to re-enter the system is the result of the existance of a deficit in hydrostatic pressure between the particles, and not of an excess osmotic pressure. In comparing the "osmotic" and the "mechanical" theories one should bear in mind that the deficit in hydrostatic pressure between the grains can exist only if the grains are kept apart by mechanical forces. Such mechanical forces imply a direct contact between particles, which seem unlikely in pure clays, unless a very advanced state of desiccation has been reached.

In studying the swelling pressures of montmorillonite, Warkington, Bolt and Miller (41) compared theoretically computed pressures based on the osmotic theory with experimental pressures. The computed pressures were based on the double layer theory and involves the determination of the electrical potential in the central plane between two parallel charged plates.

The experimental pressures were obtained by compressing a treated bentonite gel and determining the volume at various confining pressures. After reaching a predetermined terminal pressure, the specimens were allowed to swell. The process was carried through three compression cycles. The authors (41) point out that the test pressures for the first compression are higher than the subsequent cycles and attribute this to random orientation of particles. When the pressure was carried above 20 atmospheres during the first compression, the particles apparently oriented themselves to some degree in a parallel arrangement resulting in consistent results during the second and third compression cycles.

Excellent agreement between measured and computed pressures for Na-montmorillonite was obtained. The results for Ca-montmorillonite were less encouraging with the measured swelling pressures, after the first compression, being lower than the theoretical values. No conclusion is drawn as to what interparticle forces could be acting with the divalent ion and not acting with the monovalent ions. The possibility of less disassociation of divalent ions as compared to monovalent ions was not considered.

In pursuing the theory of osmotic swelling further, Ladd, 1957 (20), conducted swelling tests on compacted natural soil. Ladd also attributes the swelling, in addition to osmotic phenomena, to secondary valence or London van der Waals forces and to the effect of the negative electric field on the double layer water. In regard to mechanical or elastic swelling, reference is made to Lambe (23) who states that this phenomenon is negligible for most clays.

Based on the results of swell tests of specimens soaked in calcium chloride solutions, Ladd concludes (23), p. 24:

Furthermore, it appears that the osmotic pressure concept can satisfactorily explain a good portion of the swelling that occurs when this clay is soaked in water, particularly for samples compacted wet of optimum water content. Dry of optimum water content, however, swelling is influenced by factors in addition to osmotic pressures. These other factors may be: the effect of negative electric and London van der Waals force fields on water, cation hydration and the attraction of the particle surface for water molecules, elastic rebound, particle orientation, and the presence of air. The relative importance of these factors is unknown.

Determination of the Swelling Potential of a Clay

With the recognition of the problem of expansive soils, it has become desirable to obtain an easy method of testing a clay and then, from the results, determine if the material has the potential to swell or shrink. If the potential is there, it is then desirable to make, at lease qualitatively, a prediction of the relative magnitude of the possible volume change-stress relation. Whether or not a potentially expansive soil will or will not undergo volume changes under the existing conditions is not generally considered. Under certain conditions it is possible that further moisture content changes and, therefore, shrinking or swelling of the soil are not likely to occur.

The primary purpose of such a test is to determine if further investigation into the expansive aspect of the foundation problem is necessary.

Kanty and Brink, 1952 (15), published extensive data, collected by the National Building Research Institute of South Africa, for soils which were known to be expansive. This data consists of the Atterberg Limit values as determined for the different soils. From this data, they concluded that the Atterberg Limits can be used as a reliable guide.

The following criteria are given as indicating a potentially expansive clay.

(1) The linear shrinkage should be greater than eight per cent.

(2) The liquid limit should be greater than 30 per cent.

(3) The plasticity index should be greater than 12 per cent.

A very important contribution to the problem of identifying potentially expansive soils was made by Holtz and Gibbs (13) in 1954. From extensive laboratory testing and field experience, a criterion was presented as a guide for determining the expansive potential of a clay. This criterion is based on the colloid content (per cent minus 0.001mm), the plasticity index and the shrinkage limit,

where per cent minus 0.001mm is the per cent by weight of soil having a grain diameter smaller than 0.001mm,

the plasticity index is the numerical difference between the liquid limit and the plastic limit, and

the shrinkage limit is the moisture content obtained by assuming the remolded soil saturated when at its oven-dried volume.

In addition to the above tests, Holtz and Gibbs also investigated other tests which might be used to indicate the expansive characteristics. Included were size smaller than 0.005mm, liquid limit, free swell and montmorillonite mineral content.

The free swell test consists of slowly pouring ten cubic centimeters of dry soil passing through the number 40 sieve into a 100 cc graduate filled with water and noting the swell volume of the soil. The free swell value in per cent is equal to the final volume minus the initial volume divided by the initial volume times 100. Values below 50 indicate a soil which very seldom will exhibit volume changes even under light loads. Soil having a free swell value of 100 may exhibit considerable change under light loads according to Holtz and Gibbs. Dawson (10) points out that some Texas soils having a free swell value of 50 have caused considerable trouble due to expansion and that the limits for such a criterion should be set for each area to take into account climatic conditions.

Several (7, 10) have suggested that the "activity" value of Skempton (36) has some possibility as an indicator of the expansive properties of clays. This value is defined by Skempton as:

$$\text{Activity} = \frac{\text{plasticity index}}{\text{clay fraction}}$$

where the clay fraction is equal to per cent by weight of particles finer than two microns. This value is presumably a constant for a given soil. Skempton relates the "activity" to various properties of the clays tested, none of which were the expansive characteristics. It is generally agreed that the expansiveness of a clay increases as the plasticity index increases and also as per cent clay fraction increases. The possibility, however, that a ratio of these two properties will be any better as an indicator of expansivenss than either factor used alone is not likely. If both increase proportionally, the swell trend ratio is a constant.

Holtz, 1959 (16), extended the work which he and Gibbs had reported in 1953 (17) and based on the additional testing and field experience modified their original criteria to that shown in Table 1.

The change between this criterion and the original is very slight but probably more reliable because of the greater experience involved in arriving at the various values. The work of Holtz and Gibbs (17) is notable because it is one of the few efforts to relate the expansive properties of natural undisturbed specimens to the index properties of the soil.

Ladd and Lambe (22) working on a variety of compacted soils related several moisture conditions to swelling and swelling pressures. To indicate the expansiveness of a soil, the authors suggested a tentative system called the Potential Volume Change (PVC). This system is based on measurements of swell under a load of 200 psf, plasticity index, moisture content at 100 per cent relative humidity and the difference in volume between the field moisture equivalent and the shrinkage limit. The system relates each of these factors to the PVC value; this value is used as a measure of the degree of expansiveness of the soil. The tentative system as presented by Ladd and Lambe is given in Table 2.

This system has the advantage of relating several properties to one numerical value. The disadvantage is that only one of these, the plasticity index, is very readily obtained in all soils laboratories.

In a very extensive testing program on laboratory manufactured soils (e.g. combinations of sand and pure clay minerals), Seed, Woodward and Lundgren (35) attempted to relate the colloid content, plasticity index and shrinkage limit to the swelling characteristics of the soil. This was done to determine the applicability of the criteria

Table 1. Data for Making Estimates of Probable Volume Changes for Expansive Materials*

Data from Index Tests**			Probable expansion* % total volume change (dry to saturated condition)	Degree of expansion
Colloid content (% minus 0.001 mm)	Plasticity index	Shrinkage limit (%)		
28	35	11	30	Very high
20 - 31	25 - 41	7 - 12	20 - 30	High
13 - 23	15 - 28	10 - 16	10 - 20	Medium
15	18	15	10	Low

* Based on vertical loading of 1.0 psi.
** All three index tests should be considered in estimating expansive properties.

* After H. G. Holtz (16), p. 96.

Table 2. Classification of Soils with Respect to Potential Volume Change Due to Swelling and Shrinkage*

PVC	Category
<2	Noncritical
2-4	Marginal
4-6	Critical
>6	Very Critical

PVC	A Percent expansion of dry and/or moist sample, 200 psf load	B PI %	C Moisture content 100% humidity	D V_{FME-SL}
2	5	15	6	15
4	11	25	11	24
6	17	35	16	33

Note: (1) To interpolate, plot PVC values against A, B, C, D, values which form a straight line.
(2) Combined PVC = (2xA+B+C+D)/5

* After Ladd and Lambe (21).

proposed by Holtz and Gibbs (17) to the soils tested.

In discussing the results of these tests, the authors state (35), p. 65:

> It will be seen, as is to be expected in dealing with clays of widely varying activities, that there is no evidence of any correlation between swelling potential and percentage of clay sizes. Nor is there any evidence of a correlation between swelling potential and shrinkage limit. From these data, there would seem to be little basis for continuing to use these characteristics as indices of swelling potential. However, there is evidence of a possible correlation between swelling potential and plasticity index.

The results do indicate that for a soil of given "activity" that the expansiveness increases with increased colloid content. This would seem to indicate that the per cent colloid content is a secondary factor or a factor that has to be considered in combination with some other property. There most certainly would be agreement that a high colloid content would in general present the possibility that the soil has the potential to be expansive.

Undoubtedly this type of reasoning led the authors to the procedure of predicting swell potential on a plot of "activity" versus per cent colloids and to the equation

$$s = 3.6 \times 10^{-5} A^{2.44} C^{3.44}$$

where s = swelling potential in per cent,

A = Skempton's activity value,

C = per cent colloids,

relating the swelling potential to only the plasticity index and the per cent colloids.

The authors further show that the equation can be simplified without serious error to relate the swelling potential only to the plasticity index and arrive at the following conclusions (35), p. 87:

> In the light of the data presented, it is suggested that the proposed procedure, utilizing a knowledge of percentage of clay sizes and Atterberg Limits for a soil, provides a reasonably accurate means of predicting the swelling potential of compacted clays and that the relationship of swelling potential with plasticity index provides a good approximate method. In fact, in view of the difficulty in accurately determining the percentage of clay sizes present in a soil and the approximate nature of the relationship between the percentage of clay sizes and the quantity of clay in the soil, it is suggested that the latter method might well be best suited for practical purposes.

The authors are careful to point out that the method is not intended to predict the actual amount of swell under field conditions or as a substitute for swelling tests.

Effects of Adsorbed Ions on the Swelling of Clays

In the discussion on the mechanism of swelling, it is pointed out that one of the important factors involved in swelling is the nature of the adsorbed ion. In the ceramics industries, the use of electrolytes as additives to clay pastes to improve their workability or, in essence, to change their properties has been recognized for many years.

In 1936, Winterkorn (43) presented test results which indicate that the swelling of a soil is reduced if the pore water contains a salt solution having the same cation as that adsorbed on the clay surfaces.

Winterkorn (43) presented the results of various index tests conducted on three specially treated soils. These soils were leached and then treated with various ions. It is interesting to note that for two of the soils a marked increase in the plasticity index is noted for the Na treated specimen. The Na specimen for the third soil exhibited the least plasticity index. This soil was developed under lateritic weathering from gneiss whereas the other two were developed by podsolic type weathering from chert free limestone and from mixed glacial and loessial origin. In all cases, in the podsolic soils those properties which would indicate an expansive soil were much higher for specimens treated with sodium than for specimens treated with other ions. The opposite is true in the lateritic soil.

Swelling tests were conducted only on the podsolic soil. Specimens of powdered calcium and sodium treated soil were allowed to imbibe several concentrations of calcium chloride and sodium chloride solutions respectively. The results indicate a reduction in swelling as the salt concentration of the imbibed water increased. Winterkorn concluded (43), p. 301:

> Changes in the structure of the soil system, caused by the concentrated electrolyte solutions are indicated in the slope of the swelling-concentration curves. If the exchange ions on the soil are different from the ions in solution, the swelling-concentration relation follows no definite form. This fact is of practical importance in predicting the effect of salts on soils which vary in the amount and kind of their base saturation.

Davidson and Sheeler, 1952 (9), made a comprehensive study of the cation exchange capacity of loess and its relation to engineering properties. Although the authors were not dealing with a highly expansive soil, they did relate the cation exchange capacity to certain engineering properties which in general are indicative of a potentially expansive soil. The type of adsorbed cation held by the soil was not determined.

The results of these tests indicate that for the soils tested, an increase in the exchange capacity indicates an increase in the plasticity index. In addition an increase in the 0.002mm clay content, the liquid limit and hygroscopic moisture are indicative of an increase in the exchange capacity, all else being equal.

Goldberg and Klein, 1952 (12), conducted swelling pressure tests on Wyoming bentonite and Porterville clay treated with calcium hydroxide. The treatment resulted in a replacement of sodium ions by calcium in the soils. The results indicate a substantial reduction in swelling pressure of both soils from the treatment. At a given density the pressure for the maximum sodium hydroxide treatment is approximately one-fifth of that for the untreated Porterville clay and about one-half of that for the untreated bentonite.

In studying the effects of various cations on swelling, Hemwall and Low, 1955 (15), used soil treated with sodium, thorium and a combination of the two. As would be expected, the results compared at equal volumes indicate that as the valency or average of the valency of the adsorbed ions increases, the swelling pressure decreases. The authors attribute the results to a decreased cation disassociation from the clay for the polyvalent ion.

This phenomenon is explained very well by Taylor (37), p. 25, in discussing the influence of exchangeable ions on the double layer where he states:

> Divalent ions are more strongly attracted towards the negative surface than monovalent and therefore form a thinner layer with a higher local density of charge; this has a more effective screening action so that the forces of repulsion between the negative charges are less effective even when the particles are close together. Replacement of divalent by monovalent ions reduces the effectiveness of the screen so that the particles repel each other over greater distances. The effect of increased electrolyte concentration is more difficult to describe in these terms, but it

> has similar effect in that it results in a contraction of the space charge into a smaller volume so that it acts as a more effective screen.

Taylor also points out that in addition to valency, the influence of the adsorbed ion on the engineering properties of a soil is also dependent on the ionic radii, its polarizability and its hydration.

In studying the mechanism of swelling of compacted clay, Ladd, 1957 (21), immersed compacted specimens into 0.5 and 5.0 molar solutions of calcium chloride and distilled water. The exchangeable cations were assumed to be predominantly calcium. The swelling of the specimen decreased with the increased concentration of $CaCl_2$ in the soaking water. Based on the theory that this reduction is due to a reduction in osmotic swell, Ladd concludes that (21), p. 25:

> 3. The replacement of low valency exchangeable cations by higher valency cations (for example, calcium for sodium) can reduce swelling, since the number of exchangeable cations in the double layer is reduced.
> 4. The mixing of salt with a compacted clay can reduce swelling, since the ion concentration in the pore water is increased.

The author fails to point out in the above conclusion that it is essential that the salt used contain the same cation as those adsorbed in the clay.

White and Pichler, 1959 (42), conducted water-sorption tests on several types and combinations of clay minerals. The montmorillonite samples were selected from different locations so as to provide specimens having different exchangeable ions. Over the period tested, the sodium montmorillonite adsorbed over four times as much water as the calcium montmorillonite. A calcium-hydrogen montmorillonite adsorbed slightly less water than the calcium montmorillonite. All of the montmorillonites adsorbed greater quantities of water than the kaolinites.

The cation exchange capacity of kaolins is generally very low ranging from three to eight meq/100 grams according to Taylor (37). Under these conditions the type of adsorbed ion is relatively unimportant.

Prediction of Swell Deformation

The prediction of swelling deformations under various load conditions for large masses of soil from results of laboratory tests is essential if reliable design of foundations on expansive clays is to be accomplished. Unfortunately, the uncontrollable factors which exist in the field that affect the swelling characteristics of the soil are difficult to predict. Despite these difficulties, efforts have been made in this direction at least under certain limiting conditions.

In the literature there are several references to the swelling index, the slope of the rebound portion of the void ratio-log pressure curve. The use of this value is somewhat vague. For the conditions for which it is usually conducted, removal of load with zero pore pressure, the application to problems of structures on expansive clays is questionable.

One approach to the problem, at least to the extent of determining the pressure required to prevent swelling, is given by Means (25). The specimen is placed in the consolidometer, flooded and allowed to swell under a load equivalent to the overburden pressure. The remainder of the consolidation test is conducted in the usual manner. The pressure where the e-log p curves intersect a horizontal line indicating initial void ratio is taken as the no volume pressure. Presumably for applied loads less than the no volume change load, the heave could be computed from the difference in the initial void ratio and the void ratio at that particular load.

The double oedometer test (18) was designed to provide a means of predicting the heave of a structure. The test consists of two consolidation tests run simultaneously on identical specimens. One of the specimen is inundated and allowed to swell under a load of .01 tons per square foot; the other is sealed to prevent evaporation and allowed to consolidate under the same load. Both specimens are then loaded as in the usual consolidation test.

The e-log p curves are plotted and adjusted so that the virgin portions will coincide. The e-log p curve for the inundated specimen represents the relation between effective stress and the void ratio of the soil. The second curve represents the applied load-void ratio relation for partially saturated specimens. The difference for any given pressure represents the swell which would occur under the load if the negative pore pressure in the partially saturated soil is allowed to go to zero. From the existing pressure, the applied loads and the estimated capillary stress under field conditions, the predicted heave of the structure can be determined by the usual methods relating change of void ratio to stress imposed by the structure.

The approach to the problem is good but seems to be unnecessarily involved. The value of the constant moisture content e-log p curve is questionable. The only purpose it serves is to determine the change in void ratio from the seating load to the overburden pressure. Within the range of depths which heaving will occur, this difference in highly overconsolidated soils is very slight. Compared with the other uncertainties, it is negligible. The void ratio of the specimen prior to soaking would serve just as well.

The application of capillary stresses as an applied force might also be questioned. It does give a lower limit to the heave computation, but in addition the possibility of the surface becoming inundated should be considered as an upper limit.

The weakest point in this approach is the necessity for adjusting the two e-log p curves so that the virgin portions coincide. The authors (17) attribute the differences to local variations in moisture content. In view of generally accepted data which indicates that loading a clay soil results in some orientation of the particles, and in view of the possibility that the orientation may be effected by the moisture content at the time of loading, the justification of the adjustment needs clarifying. In other words, it has not been satisfactorily proved that if a soil is allowed to swell freely and then consolidated under a given load that it will reach the same point if it were consolidated under the load and then allowed to swell.

Of particular importance is the absolute necessity that the tested specimen not be allowed to shrink or swell prior to testing. This applies to all the methods discussed.

Clisby and Dawson (8) have proposed a method for determining the movement of structures on expansive clays based on the similarity between the rebound curve of the ordinary consolidation test and the swelling curve. The swelling curve was determined by holding an oven-dried specimen in a no volume change condition and allowing it to imbibe water. After the full swell-pressure had developed, the load was removed in decrements and the pressure determined for each step. It was found that the swell curve was parallel to the straight portion of the rebound curve of a consolidation test. The effect of oven drying on the clay used was not discussed.

A consolidation curve by desiccation was also determined experimentally. The findings show that this curve is parallel to the virgin portion of the usual consolidation curve.

To determine the movement of a structure, the authors propose conducting a consolidation test in the following manner:

(1) allow the specimen to consolidate under the overburden pressure,

(2) flood and allow to swell until completed,

(3) conduct remainder of consolidation and rebound tests in usual manner and

(4) plot the e-log p curve.

To determine the no volume change load, a line parallel to the straight portion of the rebound curve is drawn through the point of maximum swell under the overburden pressure. Where this line intersects the void ratio of the soil in the field is the no volume change load. For loads less than this, the movement can be determined from the difference in void ratios along this line for the particular load.

This approach has the advantage of requiring only a slight modification to usual consolidation tests. The similarity of the rebound and swelling curves was determined on remolded specimens. There is some question as to the validity of these findings when applied to natural deposits. It is also interesting to note that the authors make no mention of considering the loading effect of capillary water as was considered by Jennings and Knight (18).

All of the procedures reviewed are basically the same with the exception that Clisby and Dawson do not assume that the swell-pressure relation follows the consolidation curve, but show that it follows a line parallel to the straight portion of the rebound curve. There is some disagreement as to the loads considered. Jennings and Knight assume that the capillary surface will rise beneath the structure to the surface and will result in an applied force on the soil equivalent to the density of water times the distance from the point in question to the water table. The neglect of this force results in a computed heave greater than that which might occur if the force is present.

Summary of the Present State of Knowledge

Theories of Swelling

It is generally agreed that the swelling of clay soils is dependent on the

(1) type and amount of clay mineral present,

(2) type and amount of exchangeable ions adsorbed on the clay minerals,

(3) shape and to some degree the orientation of the clay minerals and

(4) nature of the pore water.

For swelling to occur the particles of a soil must be forced apart. This may result from either an increase in thickness of the adsorbed water layer surrounding each particle or by elastic expansion or bending of particles in direct contact. The latter leads to a mechanical or elastic rebound theory of swelling. At present, it is generally agreed that this phenomenon is predominant in coarse-grained soils, especially those containing high mica contents.

The increase in thickness of the adsorbed water layer may be attributed to any or all of the forces which act between the water molecules and clay mineral surface in addition to the osmosis phenomenon.

These forces as presented by Low and Lovell (24), p. 26, are:

(a) Ion-dipole attraction between exchangeable cations and H_2O molecules,
(b) Hydrogen bonding between surface oxygens and H_2O molecules,
(c) Charge-dipole attraction between exchange sites and H_2O molecules,
(d) Dipole-dipole attraction between surface oxygens and H_2O molecules due to in-phase fluctuations of negative electronic atmospheres about positive oxygen nuclei.

The relative importance of the above forces is not known although evidence presented by Hemwall and Low (14) indicates that hydrogen bonding is unimportant in swelling.

There is considerable disagreement on the ability of water molecules to link together and build up thick layers of water by dipole orientation. The fact that these layers exist is fairly well established (24), but the structure and nature are in disagreement. That this water does not have the same properties as free water is generally agreed.

The theory of swelling based on osmotic phenomenon is well established with test results indicating that for pure clay gels all or almost all of the swelling can be attributed to difference in ionic concentration between the adsorbed water and free pore water.

For mixtures of coarse and fine grain particles, the relative importance of the elastic theory and physico-chemical theories of swelling are obscure.

The question of particle orientation and its effect on the swelling properties of clay soils has been encountered by several researchers. To relate laboratory test results to natural clay deposits, it will be necessary to investigate this problem and shed more light on the importance of this phenomenon.

Prediction of Swell Potential

Of all the problems encountered in dealing with expansive clays, the prediction of swell potential has received the most thorough investigation. There is no question that by methods of X-ray defraction, differential thermal analysis, electron microscopy and ion-exchange studies, the potential for swelling can be estimated. These tests are generally unavailable to most soil mechanics laboratories, necessitating the relating of those tests which can be run in these laboratories to this property.

At one time or another attempts have been made to relate the liquid limit, plastic limit, plasticity index, shrinkage limit, per cent colloids, linear shrinkage and free swell value to the potential for swelling of a soil. As yet there is no scale or method for comparing or numerically expressing the property of swelling. This has made it very difficult to compare findings of various studies and has hampered the search for a solution to the problem.

Despite this handicap there is general agreement that the plasticity index is the most reliable, simple indicator of swell potential. There has been little attention given to the ion-exchange capacity of the natural soil as an indicator probably because of the difficulty and uncertainties in determining this property. When a simple or at least a standard procedure for determining ion-exchange capacity becomes available, the use of this value as an indicator should receive some attention.

In any case it is generally agreed that several indicators should be used and results of each compared when determining the swell potential of a soil.

Effects of Adsorbed Ions on the Swelling of Clay

It has been well established that the nature of the adsorbed ions greatly effects the ability of a soil to adsorb water with resulting swell. The relative effects of various adsorbed cations has been well established. There is also substantial evidence to indicate that the presence of certain cation in the pore water can control to some degree the swelling of a soil. This knowledge has led to one of the most significant means for possible stabilization or changing of soil properties.

The problem in exchanging ions in soil masses with assurance of permanency and the controlling ion concentrations in pore water is very complex and the progress in this area is only in the early stages of development.

Prediction of Swell Deformations

From the standpoint of the practicing foundation engineer, the prediction of swell deformations is one of the most significant problems encountered in dealing with expansive clays. It is also one of the most complex.

At present the most reliable approach to this problem is the conductance of swell tests in the laboratory on undisturbed specimens initially at their field moisture content. The results of these tests are reliable only to the extent to which the test conditions duplicate field conditions. There is insufficient data for most areas to relate climatic conditions, water table fluctuations, effects of changing surface conditions and possible changes in the chemical nature of ground water to the results of laboratory tests.

Work on undisturbed natural soils has been neglected to a large degree, and the swell behavior of soil masses under various conditions is obscure.

CHAPTER III

PURPOSE OF THE RESEARCH

The purpose of this investigation is to study the swelling and shrinking mechanisms of the Yazoo Clay, to extend the present state of knowledge on expansion phenomena and to clarify some of the factors involved. Moisture content-swell pressure relations, the effect of high salt concentration in the pore water on swelling, relative vertical and lateral swelling pressures, and relative vertical and lateral shrinkage of undisturbed specimens will be studied in an effort to clarify the importance of various forces acting in the swell phenomenon.

Statement of the Problem

The problems resulting from the volume change of expansive clays can be divided into three general groups:

(1) the identification of potentially expansive soils with some indication as to magnitude of volume change which can be expected;

(2) the quantitative prediction of swelling deformations, swelling pressures, and the conditions under which they will occur; and

(3) the design of foundations to withstand the predicted deformations and pressures.

The third group is one of design and is not considered in this study.

The first and second groups are very closely related and can not be entirely separated. Those properties of a clay which make it possible to correlate the expansive potential to physical tests are also the properties which are responsible for the swelling phenomena.

In general, past research has shown that the expansive nature of a clay soil can be attributed to mineralogical content (both type and quantity), the type and quantity of adsorbed ions present on the clay mineral, the moisture content and the nature of the pore water, the soil structure and its previous stress history. Any factors such as temperature or pressure which may affect the properties of the clay mineral, adsorbed ions or the adsorbed water will also affect the expansive characteristics of the soil. The problem is determining the relative importance of the various interactions and why and under what conditions they occur.

Objectives

The rather limited knowledge of the mechanics of clay expansion, especially on natural undisturbed soil specimens, necessitated the initiation of this investigation under broad objectives. This broad objective was to clarify where possible the discrepancies which exist between the various theories on clay swelling and to determine and explain the pressure-deformation-moisture content relations of the Yazoo Clay.

As with any research program, the preliminary testing resulted in the development of what might be called secondary objectives. Obviously, all of the questions and side issues which arise during the process of conducting a research program can not be thoroughly investigated. A few of these were taken through various degrees of investigation and are offered as secondary objectives. In essence, these objectives when considered together make up the primary purpose of this study. They are to:

(1) determine under what moisture and pressure conditions the Yazoo Clay will expand and to study the long time swelling characteristics of the Yazoo Clay so as to better judge the methods presently used to measure swelling,

(2) relate the shrinkage characteristics of the Yazoo Clay to the swelling,

(3) study and explain the relation of vertical to lateral swelling pressure,

(4) study and explain the anisotropic effect possibly due to preferred orientation of the mineral grains on swelling and shrinking behavior of the Yazoo Clay,

(5) study the relative importance of the various factors which are generally attributed to be the cause of swelling, and

(6) arrive at a new or recommend an existing method for expressing the expansion or shrinkage characteristics of either remolded or undisturbed soils.

CHAPTER IV

THE YAZOO CLAY

The Yazoo formation is the upper part of the Jackson group. This group crops out in a belt across central Mississippi from Yazoo County on the west to Clark and Wayne counties at the Alabama state line. This belt varies in width from approximately 35 miles on the west to less than ten miles on the east. This includes the City of Jackson, the most heavily populated area in the state. Fortunately, much of the city is founded on alluvial deposits of the Pearl River and its tributaries. Between these narrow alluvial plains, the Yazoo is covered with a thin weathered loess. This deposit is, in general, not thick enough to provide cover beneath foundations which will prevent changes in moisture due to either climatic changes or other environmental surface changes.

The presence of such a material as the Yazoo Clay in a rapidly expanding metropolitan area presents a serious foundation problem. Although not as publicized, the problem is just as serious in many smaller towns throughout the out-crop belt and on both the state and federal road programs.

The thickness of this formation varies considerably from 500 feet in Warren County to approximately 100 feet at the Alabama state line (6). In any case, the depth is sufficiently great to make impractical the extending of foundations to a depth below the Yazoo Clay.

Geologic Origin

The Yazoo Clay is considered to be of the Eocene Age. According to Monroe (26), it appears to have been formed by deposition close to shore of clay brought into the sea by a large or by several rivers. The soft marly limestone which is sometimes seen in the Yazoo Clay is probably the result of deposition during times when the clay content of the sea was less than usual.

Physical Description and Properties

The Yazoo Clay varies from a blackish, blue-gray, calcareous clay in the lower portion to a more greenish less calcareous clay near the top (26). The fresh clay will weather rapidly into an olive-gray and then into a buff or yellow-tan material. At the location where the samples for this investigation were secured, the yellow-tan weathered clay is from 30 to 50 feet thick overlying a layer of dark blue-green clay. The value of 30 feet is more typical of the area.

The weathered material used in this study contains considerable lenses of gypsum. Mellen, 1940 (26), reports that gypsum is not found below the zone of oxidation or in the zones of severe weathering. It is thought that gypsum results from the effect of decomposition of pyrite on calcareous fossils (26). Gypsum crystals below a depth of 30 feet are uncommon (27).

Berquist (3) reports comparisons of X-ray patterns of clay from the Yazoo with patterns from Wyoming bentonites. The comparisons indicate that montmorillonite is the major clay mineral present.

Buck, 1956 (5), using the minus two-micron portion of samples obtained a few miles southwest of Jackson near the top of the Yazoo, conducted X-ray defraction studies and reported the following mineralogical composition: kaolinite group, 45 per cent; montmorillonite, 30 per cent; illite, 15 per cent; and non-clay minerals, 10 per cent. The large per cent of the kaolinite group is surprising in view of the total lack of any mention of the presence of kaolinitic minerals in either the Yazoo or Scott County Geological Survey Bulletins (2, 25).

Clark (6) reports the physical properties of the Yazoo Clay to be: plastic limit, 18 to 26; liquid limit, 85 to 126; plasticity index, 59 to 98; and shrinkage limits, 9 to 11. The values of plastic limit are somewhat lower and the liquid limit values higher than those obtained on tests of samples for this study. Clark does not indicate it, but the values cited are most likely for unweathered or only slightly weathered clay.

Redus, 1962 (31), reports values for the liquid limit varying from 50 to 120, and for the plasticity index, from 30 to 80. The lower values are probably more typical for the weathered clay.

Chemical Properties

Chemical analysis of the Yazoo Clay are reported by McCutcheon (26, 3) in the Geological Survey Bulletins for Yazoo and Scott Counties. Typical analyses from these publications are shown in Table 3.

Sampling

The samples for this study were obtained in an open pit at the lightweight aggregate plant of Jackson Ready-Mix Concrete, Jackson, Mississippi. The samples were obtained in two groups, both secured as

Table 3. Chemical Properties of Unweathered Yazoo Clay

Constituent	Scott County*	Yazoo County**
Ignition Loss	23.35	17.17
Silica SiO_2	30.45	41.58
Alumina Al_2O_3	12.31	14.92
Iron Oxide Fe_2O_3	5.16	7.15
Titania TiO_2	0.47	.75
Lime CaO	25.00	18.03
Magnesia MgO	0.12	.60
Manganese MnO_2	0.09	--
Alkalies K_2O, Na_2O	0.29	.34
Sulfur SO_3	1.88	2.10

* After Bergquist and McCutcheon (3).
** After Mellen and McCutcheon (26).

near the same location as possible, at a depth of approximately 20 feet below the original ground surface in the weathered clay and approximately one and one-half feet from the face of a new cut bank.

The sampling was done in four-inch diameter thin-walled tubes eight inches long. The tubes were hand driven into the clay with the aid of a sledge hammer and driving device. The large tube was used so that ample trimming could be made in the laboratory to eliminate, as much as possible, the disturbance due to driving the tube into the soil. All of the samples for each group were secured at approximately the same elevation and within an area about two feet wide and eight feet long. The tubes were removed by digging them up with a shovel, capped at each end with aluminum foil and placed in a plastic bag.

At the laboratory, the samples were left as prepared in the field and placed in a moist room.

CHAPTER V

INSTRUMENTATION

Pressure Cell

One of the objectives of this investigation was to study the relation of vertical swelling pressure to lateral swelling pressure. To achieve this, a special pressure cell was required. In addition to the pressure requirements, the cell was designed so that predetermined amounts of vertical swelling could be allowed without affecting the lateral confinement and pressure measurement.

Swelling tests are generally conducted in the ordinary consolidometer. There is some question as to the effect of the rigid confining ring on the vertical movement of the specimen. To eliminate this restraint and to provide a sensitive method for measuring lateral pressure, a fluid confining system was adopted.

To make the measurement of the vertical pressure independent of lateral pressure and to prevent any effect on the lateral pressure by vertical deformation, it was necessary for the loading piston to be the same size as the specimen to be tested. The large piston complicated the problem of providing friction-free movement with no leakage and a minimum of space which might trap air in the confining chamber.

One of the most serious problems was in devising a method in which the confining fluid could be introduced with a minimum of trapped air.

Cell Construction

To meet the above requirements a special cell was constructed in the machine shop of the Georgia Institute of Technology Engineering Experiment Station. The cell consists of a heavy brass base, a bronze confining cylinder, a brass top and bronze piston. The top of the piston is fitted with a socket to receive a 0.75 inch stainless steel ball for transmitting the vertical swelling pressure to the measuring yoke.

To facilitate flooding the ends of the samples, two fittings were provided in the base and the load piston. These fittings provide for passage of water to porous stones at the upper and lower surfaces of the specimen. The dual system was adopted so that water could be introduced through one fitting and allow the air to escape through the other, increasing the probability that immediate complete wetting of the ends of the specimen occurred. Four brass machine screws were used to clamp the top and the base onto the confining cylinder. A thin, hard rubber gasket is used to prevent leakage between these parts.

The fit between the load piston and the top was one of the most critical aspects of the cell construction. This fit had to provide, as near as possible, friction-free movement of the piston with no leakage. To accomplish this, a close tolerance machine fit was made with a rubber o-ring provided near the lower surface of the top part of the cell to prevent leakage. The o-ring groove was designed just large enough to accommodate the o-ring and not cause the ring to jamb the piston. To reduce the chance of air being trapped around the o-ring, which would allow expansion room for the confining fluid, a vacuum grease was used to fill the void between the ring and the groove.

Fittings were provided at mid-height of the cylinder, on opposite sides, to accomodate a pressure measuring device and to provice a means for filling and draining the confining chamber.

The design of the cell was made heavy to reduce the deformations in the cell during testing to a minimum. Figure 1 shows a cross section of the cell and the principal accessories. The partially assembled cell is shown in Figure 2. Figure 3 shows the cell ready for testing with the temperature control chamber and strain indicator. Neglecting the effect of restraint on the end of the bronze cylinder, a fluid pressure of 100 pounds per square inch results in a deformation of the cell and compression of the glycerin equivalent to a change in specimen diameter of 0.0009 inches.

Vertical Swelling Pressure Measurement

It was not only necessary to measure the swelling pressure, but was also necessary to be able to keep the vertical deformation to a minimum or at some predetermined amount. To do this, a heavy steel beam was provided with a steel ball to match the socket on the ram. The ends of the beam were connected to the base by two one-inch aluminum rods. Support was provided so that the beam weight was taken by the rods and not by the specimen.

Each rod was machined down to a rectangular shape having an area of 0.16 square inches for a distance of one inch. Two 0.5 inch epoxy back foil SR-4 gages were placed on opposite sides of the rectangular sections. The two gages of each rod were placed in series and the

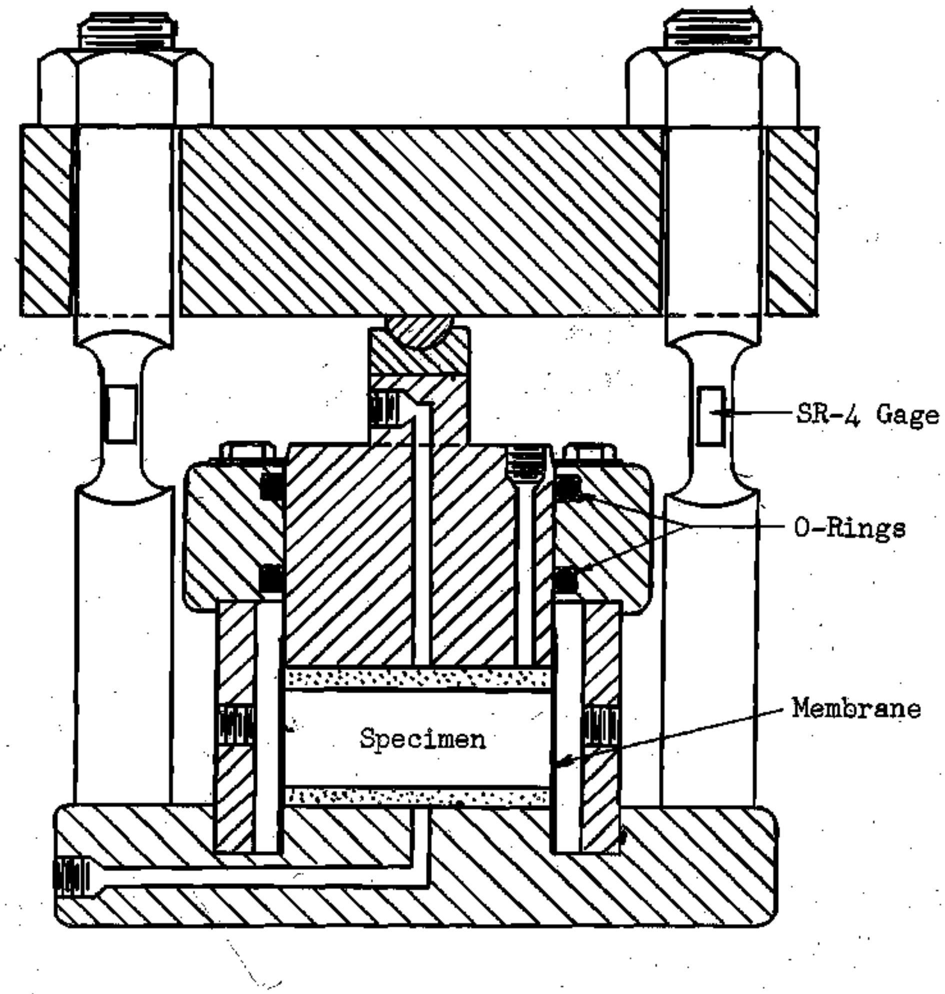


Figure 1. Cross Section of Swell Pressure Cell.

Figure 2. Pressure Cell, Disassembled.

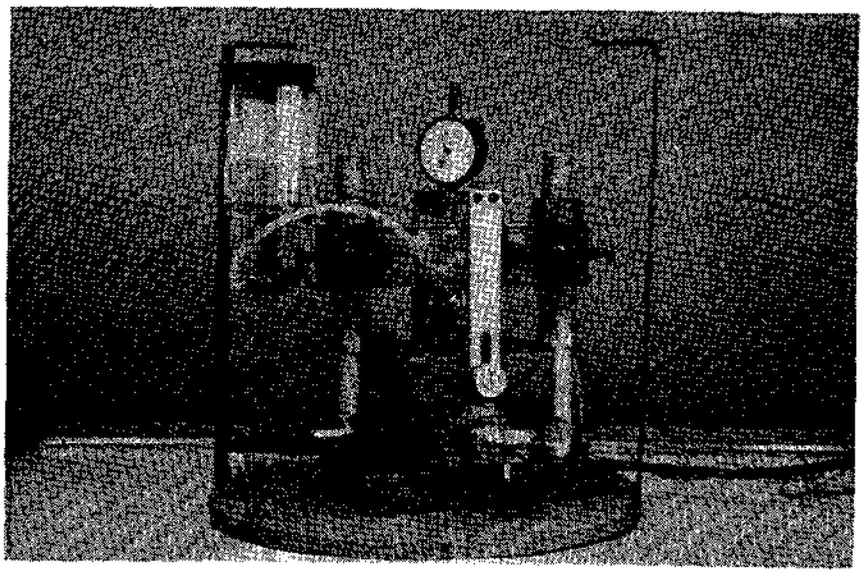


Figure 3. Pressure Cell, Assembled.

pairs then placed in parallel. A dummy gage was provided on a separate aluminum bar. The system was connected as an internal bridge to an SR-4 indicator.

The system was calibrated by introducing air pressure into the cell so that it acted on the ram. Data for pressure versus strain reading curve was obtained. The relation was found to be linear.

Lateral Pressure Measurement

A fitting to receive a Dynisco pressure transducer was installed in the confining cylinder. The transducer was connected to the external bridge poles of the same indicator used for the vertical pressure measurement. The gage factor was set to accommodate the vertical system so that it was necessary to obtain a calibration curve for the transducer. This was accomplished by using measured air pressure acting on the confining fluid in the cell.

Both the transducer and the vertical system were left connected to the indicator throughout a given test. The only switching was from the external to the internal bridge circuit within the indicator.

Deformation Measurements

To measure the vertical extension of the specimen, a yoke attached to the top of the cell holds a dial indicator which is placed in contact with the beam directly over the center of the ram.

Lateral deformation of the specimen is controlled by the characteristics of the cell. A curve of lateral expansion versus pressure was obtained by assembling the cell with a dummy specimen. The confining fluid (glycerin) was introduced through a high pressure plastic tube; and, by applying air pressure in the tubing and measuring the movement of the surface of the glycerin, fluid volume change versus pressure data were obtained. The volume change data were converted to equivalent change in diameter of a specimen, and a curve of unit deformation versus pressure plotted.

Several sets of data for independent assemblage of the cell were taken. For pressure above approximately four pounds per square inch, it was noted that the curves were parallel. By using four psi as a starting point for the test, one curve could be used to determine the lateral deformation of the specimen.

During the latter tests two dial indicators were installed through the sides of the confining cylinder. O-rings were used on the indicator shafts to prevent leakage, and a spring held the gage foot against the specimen and prevented the fluid pressure from forcing the gage outward. The dial indicator data were not entirely satisfactory because of consolidation beneath the gage foot after the initial swelling had occurred. The initial data were in fair agreement with the calibration curve.

Temperature Control

To prevent temperature changes from producing pressures in the closed confining system, a constant temperature chamber was constructed. This chamber consisted of a cylindrical water jacket. A refrigeration unit was used to circulate water through the jacket at a temperature of 20 degrees centigrade, plus or minus one degree. The cell was placed in the chamber at least six hours prior to testing so that a constant temperature could be reached. The fluctuation of the coolent temperature over the two degree range was so rapid that the air temperature within the chamber remained constant for all practical purposes.

Operation

To place the specimen in the cell, it was first placed between two porous stones and encased in a rubber membrane 2.5 inches long. With the ends of the membrane folded back on the stones, the stones were centered on the base and the loading piston placed on the top stone. The membrane was then rolled into position on the base and the piston. To prevent leakage around the membrane, a heavy rubber band cut from a bicycle innertube was used to secure the membrane to the base and piston. With the gaskets in place, the confining cylinder was placed in the recess provided. The top was lowered over the piston being careful not to disturb the specimen and bolted to the base. The bolts were tightened in such a manner that a uniform pressure existed on the gaskets between the top, the base and the cylinder.

To fill the confining chamber the cell was turned on edge so that the fittings in the cylinder were on a vertical line with the pressure transducer fitting on top. Glycerin was introduced at the bottom from a reservior by means of a plastic tube. All connections must be tight. The glycerin was allowed to rise to the top surface of the transducer fitting, the transducer installed and the cell righted.

The vertical pressure measuring yoke was installed with supports to prevent the weight of the yoke beam from resting on the piston. The dial indicator for measuring vertical deformation, the transducer and the water lines were then connected.

The cell was lowered into the constant temperature chamber and allowed to reach the temperature of the chamber. During this time, a pressure of four psi was maintained on the glycerin by means of an air pressure regulator. By keeping the confining chamber valve open and the vertical pressure yoke clear of the piston, no build-up of pressure due to the changes in temperature could result.

When the temperature in the chamber became constant, the confining chamber valve was closed and a lateral pressure reading made with the initial four psi pressure which is on the specimen. A zero reading is taken on the vertical system and the cross beam brought into contact with the piston. All dial gages were then zeroed.

The test is started by opening the valves in the water lines connecting the reservior to the base and the piston.

Consolidometers

Several of the testing programs were conducted in consolidometers which are a permanent part of the Georgia Institute of Technology Soils Laboratory. Two sizes of consolidometers were used.

The 2.375 inch consolidometers were used for all consolidation and the double oedometer type (18) tests. These are semi-fixed ring type consolidometers, utilizing a lever loading system with an ultimate capacity of 80,000 pounds per square foot pressure on the specimen. The consolidometer ring is stainless steel and serves both as a cutting ring and the confining ring. To prevent evaporation during tests in which the sample is not flooded, the interior portions of the consolidometers were wetted and the device sealed with a polyethelene membrane.

For making comparison studies, a miniature (1.40) consolidometer was used. For most work a consolidometer of this size is not desirable because of the large portion, as compared to the total specimen, which is disturbed to some extent by trimming. For this study it was thought that this disadvantage could be kept to a minimum and was less significant than the advantage of being able to obtain as many as twelve specimens from one tube sample. In addition, by making direct comparisons within a given series, the effect of disturbance is minimized.

The construction of these consolidometers is similar to the 2.375 inch ones with a specimen size of 1.40 inches in diameter by 0.75 inch in height. The load capacity if 10,000 pounds per square foot, but for this study it was extended to 12,500.

CHAPTER VI

TESTING PROGRAM

To accomplish the objectives of this study, a rather broad testing program was undertaken. In general, the testing falls into two categories--one, to determine the physical and engineering properties of the soil and second, to produce data relating the various factors which cause or result from the swelling and shrinking of the clay. The first group is primarily used to identify the soil, to provide a method of comparing results with other published data and to provide a means for relating the findings to well-known engineering properties. These are for the most part standard soils laboratory tests.

The second group consists of special tests or modifications of standard tests to provide a means for determining the behavior of the specimen under a controlled condition.

Control and Identification Tests

Standard Laboratory Tests

Liquid limit, plastic limit, shrinkage limit, moisture content, specific gravity and grain size analysis were made of selected specimens from various samples. As previously described, all of the samples came from within a very small area at approximately the same elevation. Due to this very close proximity of samples, complete testing of each sample was not necessary. Liquid and plastic limit tests were conducted on all samples used for swelling pressure determinations. Grain size analyses were conducted on each sample used for ion-exchange capacity determinations. The specific gravity tests were conducted on selected samples to obtain an average value to use in all computations. The variation of specific gravity values was negligible. These tests were conducted according to standard procedures as given by the American Society for Testing Materials (1).

Special Tests

Test to Determine Swell Pressure (Both Vertical and Lateral)

The purpose of this series of tests was to obtain data relating the initial moisture content of the specimen to swell pressure which would develop when the sample was allowed to imbibe water freely. The lateral deformation was maintained as near zero as possible with vertical deformation allowed to occur to amounts varying from near zero to the maximum.

Sample Preparation. Disks approximately two inches high were cut from the cylindrical samples, wrapped in aluminum foil or placed in polyethelene bags. These wrappings were not sealed so that the sample would slowly dry. The rate of drying was controlled to prevent cracking of the soil and to insure a more even distribution of moisture within the soil. Drying time depended on the desired moisture content. Prevention of cracking was not entirely successful and many samples had to be discarded for this reason.

Specimens were trimmed from the disk by means of a stainless steel cutting ring having an inside diameter of 2.80 inches and a height of 1.00 inch. The ring was centered on the soil disk and a light pressure applied. The sample was trimmed to a diameter slightly larger than the ring, and the ring pushed onto the soil until the soil protruded from each end. Trimming of the end was done with a knife and straight edge. To prevent uneven advancement of the ring, a spring tester was converted into a small press with a rotating table to facilitate trimming. The specimen was removed from the cutting ring, weighed, measured, installed in the pressure cell which was then placed in the constant temperature chamber.

Test Procedure. After the cell reached the temperature of the chamber, initial readings for vertical and lateral pressure were made. The tests were started with zero vertical pressure and four pounds per square inch lateral pressure.

After zeroing the dial gages, the valves connecting the water reservior to the specimen were opened allowing the specimen to imbibe water freely.

Pressure and deformation readings were made initially at short time intervals. After the first 24 hours, at least two readings were made each day.

When the increase in pressure with time stopped or became equal to about one pound per square inch per day, the test was ended. Usually this required from one to several days depending on initial moisture content of the specimen.

The specimen was then removed from the cell, measured, weighed and dried in the oven at 105 degrees centigrade. Weight and measurements were taken after drying had been completed.

Long Time Swell Pressure Tests

It was noted in the moisture content-pressure series of tests that after the initial pressure build-up, the rate of increase dropped off but still showed a slight increase each day for several specimens. To determine if this low rate of pressure increase could become significant, several tests were conducted allowing the test to run for many days.

The first of this series was allowed to run for 43 days at which time a leak in the system developed and the test was ended.

The second test was allowed to run for 94 days at which time the test was terminated because a definite drop in the rate of pressure increase became apparent.

Sample Preparation and Test Procedure. The sample preparation and test procedure for these tests was identical with that of the moisture content-pressure series.

Test to Determine the Effect of Soil Structure on Swelling and Consolidation

This test was composed of four series of tests, two utilizing the 1.4 inch consolidometers and two using the 2.375 inch consolidometers. The 1.4 inch series consisted of eight and six specimens with the 2.375 inch series having four specimens in each. In each series half of the specimens were orientated so that the bedding plane of the natural soil was horizontal in the consolidometer. The second half of the specimens was orientated with the bedding plane in a vertical position.

Sample Preparation. As the soil sample is ejected from the thin-walled sampler, it is in the form of a cylinder 3.87 inches in diameter and approximately six inches long. To prepare the 1.4 inch specimen, two circular disks approximately 1.5 inches high were cut from the center of the samples. Two quarters of each disk were used to make up a set of four specimens.

Specimens with the bedding plane horizontal were cut by placing the cutting ring on the flat surface of a quarter disk as near the point as possible. The sample was trimmed and the ring pushed onto the specimen. The ends were trimmed by means of a wire saw and straight edge.

Specimens with the bedding plane vertical were trimmed by cutting a flat section from the pie-shaped quarter disk so that the axis of the cutting ring coincided with a radial line of the original sample. The trimming was done as before.

The 2.375 inch specimens were prepared by cutting from the center of the tube sample a disk approximately three inches long. The two end disks were used to make a set of two specimens having the bedding plane horizontal. The center section was cut in two through the axis of the sample and a specimen made from each half. These two specimens had the same axis which coincided with a diameter line of the original sample.

Two cubes approximately 1.25 inches on a side were trimmed from the remaining undisturbed pieces of each sample. These cubes were measured, weighed and allowed to air dry under room conditions to determine the shrinkage properties.

Moisture tests were conducted on trimmings from each sample. Generally, one determination was made for each disk.

Test Procedure - 1.4 Inch Tests. The specimens were placed in the consolidometer with porous stones on top and bottom. A plastic membrane was used to seal the consolidometer to prevent moisture loss during the first loading.

A seating load of 45 pounds per square foot was applied and the dial indicators zeroed. The seating load was then removed and the first load of 500 pounds per square foot applied. The specimen was allowed to consolidate at constant moisture content under this load. Time-deformation data were taken.

The specimens were then flooded and allowed to swell. Upon completion of the swelling loads of 1000, 2000, 4500, 7000, 10,000 and 12,500 psf were applied at time intervals ranging from 24 to 48 hours. Unloading was made to 7000 and 500 psf for the rebound curves. The soil specimens were removed from the ring, measured with a micrometer caliper and allowed to air dry. The samples were remeasured and placed in an oven.

Test Procedure - 2.375 Inch Tests. The specimens were placed in the consolidometers. Exposed surfaces within the consolidometer were moistened and the consolidometer sealed by means of a plastic membrane. The dial gages were zeroed under a seating load of 100 pounds per square foot.

In one of these series, the loading was done in increments of 500, 1000, 2000, 4000, 8000, 16,000, 32,000, 64,000 and 80,000 pounds per square foot. The specimens were not inundated and evaporation was prevented by the above mentioned plastic membrane. Rebound loads of 16,000, 4000 and 500 pounds per square foot were used. The specimens were removed from the consolidometer rings, measured, weighed and oven dried. They were then remeasured and weighed.

The second of this series was conducted with the same load increments except that the specimens were flooded after consolidation had been completed under the 500 psf load. After completion of the rebound to the 500 psf load, the specimens were removed and treated as before.

Linear Shrinkage Tests

The purpose of the linear shrinkage tests was to determine the magnitude of the shrinkage parallel to the vertical axis of the soil from the moisture content at or near the plastic limit to the air-dried condition under a nominal load.

To achieve somewhat similar conditions to those that occur in the field, it was first decided to set up the tests in the 2.375 inch consolidometers so that drying from the sides could not occur until the specimen had shrunk away from the consolidometer ring. Actually, this is a limiting condition which does not necessarily occur in the field. Soil confined by additional soil may lose moisture laterally if a moisture gradient occurs between the soil masses in question. To increase the number of tests which could be run, it was decided to trim the specimen using the 2.80 inch cutting ring and run the tests without the confining ring. There was no indication of any lateral expansion of the specimen due to the nominal load used. During the shrinkage period, the specimen was subjected to a vertical load of 100 pounds per square foot by means of a 1.4 inch diameter plate placed at the center of the top surface of the specimen.

Specimen Preparation. The specimens were trimmed, using a 2.80 inch cutting ring, from undisturbed samples so that the bedding plane of each specimen was perpendicular to the direction of loading. The ends of the specimens were trimmed with a wire saw and straight edge to the same height as the cutting ring, 1.00 inch.

The necessary data for making moisture determinations and void ratio computations were taken.

Test Procedure. The specimens were placed on a flat surface in the 1.40 inch consolidometer loading frame. The 1.4 inch load plate was centered on the specimen, the loading arm brought into position and the dial gage zeroed. The 100 pounds per square foot load was then applied and time-deformation data taken. Drying was allowed to proceed until no appreciable vertical deformation was detected. At this time the specimens were removed from the ring, weighed, measured and placed in the oven to dry. The specimens were again weighed and measured after oven drying.

To determine the vertical shrinkage, the five-minute reading was subtracted from the final reading.

Vertical and Lateral Shrinkage Comparisons

It was discovered early in this study that the natural clay specimens did not shrink uniformly in all directions. Closer observations indicated that the vertical shrinkage was much greater than lateral and that the shrinkage in the two horizontal directions was not significantly different.

Vertical and lateral linear shrinkage tests were conducted to determine the ratio of vertical to lateral shrinkage of the undisturbed soil and to compare it with the ratio for remolded specimens. Some of these tests were conducted in conjunction with the tests to determine the effect of soil structure on swelling. Other parts of the data included were obtained from the measurements of specimens used in other tests.

These tests consist of measuring a geometric chunk of soil, allowing it to dry and measuring it again at the same points. Cracking made it necessary to discard some of the specimens.

Preparation of Specimen. Specimens were cut into either cubes approximately 1.5 inches on a side or into cylindrical prisms 1.4 inches in diameter and from 0.75 to about 1.50 inches high.

Procedure. The specimens were numbered, weighed and measured. Measurements were made with a set of machinest micrometers at marked points. They were allowed to air dry in an open pan, weighed and remeasured. Some specimens were then oven dried and additional measurements taken.

Double Oedometer Test

The double oedometer test was discussed in Chapter II as a means of predicting the movement of structures on expansive clays. Because it has been widely discussed and has received some acceptance, several of these tests were conducted on the Yazoo Clay in order to study the problems involved and to determine as much as possible the validity of the test.

A question was raised in Chapter II pertaining to the validity of adjusting the e-log p curve to make the virgin portions coincide. This adjustment is based on the assumption that the e-log p curve of a given soil has a characteristic shape regardless of the water content at the start of the consolidation test. These tests were conducted to study this question.

Sample Preparation. The specimens for each test (two) were obtained from adjacent disks sliced from the cylindrical samples. Each was trimmed to fit the consolidometer ring in the same manner as previously described in the other tests which utilize the 2.375 inch consolidometers.

The specimens were placed in the consolidometer on air dry stones and with the interior portions of the consolidometer moistened and sealed with a plastic membrane.

Test Procedure. A load of 100 psf was applied to the specimens and the dial gages zeroed. One of the specimens was flooded and allowed to swell for 24 hours. At the completion of the swelling, both specimens were loaded in the usual sequence with load increments being applied at 24-hour intervals.

Data for plotting the e-log p curve were obtained.

Ion Exchange Capacity

Tests were conducted on several selected samples to determine the ion exchange capacity of the natural soil. In each case either two or three determinations were made for each sample. The samples were selected to cover the widest possible variation in the plasticity index of the soil.

Sample Preparation. To prepare the specimens, selected samples of soil were air dried under room conditions and pulverized to pass through a number 100 sieve. It was possible to grind the soil so that all passed through the sieve.

Test Procedure. The test procedure used was adopted from Davidson and Sheeler (9). This procedure consists of leaching a ten gram specimen in a filter tube with ammonium acetate solution. The soil then is leached with methyl alcohol, transferred to a Kjeldahl flask, covered with distilled water and a teaspoon of magnesium oxide added. The mixture is distilled through 50 ml of 0.1000 N hydrochloric acid containing methyl red solution.

After distillation is completed the hydrochloric acid is titrated with 0.1000 normal sodium hydroxide.

The cation exchange capacity is calculated from the equation:

$$C = \frac{A - B}{d} \times 100$$

where C = cation exchange capacity in milliequivalents per 100 grams of soil,

A = (ml of HCL used) (normality of HCL),

B = (ml of NaOH used) (normality of NaOH) and

d = oven dry weight of sample in grams.

Test to Determine the Effect of Calcium and Sodium Chloride on the Swelling of Undisturbed Samples

This series of tests was conducted to obtain data relating the salt content of imbibed water to the magnitude of swell and to determine the importance of osmotic swelling in undisturbed specimens of the Yazoo Clay.

The nature of the adsorbed ions were not determined but with the highly calcareous nature of this clay, it is most likely that they are calcium ions. Based on this assumption several series of specimens were immersed in solutions of calcium chloride. In each series, distilled water and several concentrations of the calcium chloride solution were used so that relative swelling could be determined.

Due to the uncertainty of the nature of the adsorbed ion, a second series of tests was conducted in which the specimens were allowed to imbibe several concentrations of sodium chloride solutions as well as distilled water.

It was thought that if undisturbed specimens could be electro-dialysized and then subjected to solutions of sodium chloride and calcium chloride, a better understanding of the reaction between salt solutions and the swelling characteristics could be obtained. According to Grim (12), electro-dialysizing a soil can cause decomposition of montmorillonite minerals, if present, and can result in the loss of what are usually unexchangeable ions. Because of these uncertainties, this approach was abandoned.

Sample Preparation. To prepare the specimens for this series of tests disks approximately 1.5 inches thick were cut from the tube samples. The disks were quartered and specimens for the 1.40 inch consolidometers cut from each quarter. The bedding plane of the natural soil was kept in a horizontal direction in the consolidometers. The trimming of the specimens was similar to that for the specimens for the series of tests on the effect of soil structure on swelling. To assure random results, the order of trimming was done so that two adjacent specimens would not make up a pair subjected to the same solution.

Height and weight data were obtained for each specimen. Moisture determinations were made on trimmings of each.

Testing Procedure. Table 4 shows the number of specimens for each series for each concentration of imbibed solution.

In each series the specimens were placed in the consolidometer with porous stones at the top and bottom. With no applied load, the dial gages were zeroed; then, a load of 500 pounds per square foot was applied. A deflection reading was made at one minute, after which the specimens were inundated with appropriate solution. Dial readings were made at various time intervals over a period of at least five days.

X-Ray Diffraction Tests

X-ray diffraction studies have been instrumental in the identification of the many clay minerals and in the determination of mineral structure. Such studies are generally made on selected specimens to eliminate the complications involved in interpreting X-ray data from soils containing several different clay minerals. These tests were undertaken to see if it is feasible to use X-ray studies on untreated specimens as a means of identifying the soil as being expansive. With the entire study being restricted to one clay, the data will be inconclusive but may at least point out the problems involved.

Two tests were conducted to study the preferred orientation of the particles.

Sample Preparation. Four tests were conducted on powdered specimens prepared in the following manner.

Specimen 1 was taken from one of the tube samples.

Specimen 2 was obtained from a sample of the unweathered Yazoo

Table 4. Number of Specimens Immersed in Salt Solution

	Molar Concentration						
Series	0*	1	2	3	4	5	6
				Calcium Chloride			
1	1	1				1	
2	1					2	
3	2			3			3
4	2					3	
5	2		2		2		2
				Sodium Chloride			
6	2		2		2		2
7	2		2		2		2
8	2		2		2		2

* Distilled Water

Clay taken at a greater depth than the usual test samples.

Specimens 3 and 4 were obtained from tube samples of the weathered clay. The samples were separated by means of sedimentation and that portion finer than two microns used for Specimen 3. Specimen 4 was secured from the coarser fraction of the sample.

Each of the specimens was air dried and pulverized to pass a no. 300 mesh sieve. The powder was lightly pressed into the specimen holder for testing.

Specimens 5 and 6 were cut from an undisturbed sample of the weathered clay. Specimen 5 was cut so that the flat surface of the specimen was parallel to the bedding plane of the soil. Specimen 6 was trimmed so that the flat surface was normal to the bedding plane. The specimens were slightly moist when tested.

The defraction patterns were run with copper Kα radiation.

CHAPTER VII

RESULTS

Soil Index and Classification Properties

The results of the index and classification tests conducted on the samples of Yazoo Clay used in the testing are shown in Table 5.

The variation in liquid limit and plasticity index was very pronounced over very short distances. It is possible, by these values, to group the results into two parts. One group had a liquid limit ranging from 64 to 65 and a plasticity index ranging from 33 to 38. The second group had a liquid limit ranging from 68 to 72 and a plasticity index ranging from 38 to 42.

Initial Moisture Content vs Swelling Pressure

Sixteen tests were conducted in the swell pressure device in which the only deformation allowed was that of the cell. The vertical strains ranged from zero to 0.7 per cent while the lateral strain ranged from 0.1 to 0.8 per cent depending on the swell pressure developed. The initial specimen data is shown in Table 6 and test results in Table 7.

Although the specimens were secured within a very small area (several square feet), the variations in the index properties are considerable with the exception of the plastic limit. Using the liquid limit and plasticity index as a criterion, the specimens were divided

Table 5. Soil Index Test Results

Specific Gravity	2.73
Liquid Limit	64% to 72%
Plastic Limit	27% to 32%
Plasticity Index	33% to 42%
Shrinkage Limit	10% to 16%
Moisture Content (when sampled)	28% to 38%
Percent finer than 2 micron	53% to 62%

Table 6. Data for Swell Pressure Test Specimens

Test no.	Initial moisture content	Initial void ratio	Degree of saturation	Liquid limit	Plastic limit	Plasticity index	Shrinkage limit
1	22.7	.712	87	65	27	38	13.9
2	27.0	.776	93	--	--	--	--
3	17.0	.678	69	65	32	33	--
4	29.5	.828	95	65	31	34	10.3
5	30.7	.908	93	64	26	38	13.2
6	9.2	.578	43	64	26	38	13.2
7	16.3	.708	--	65	31	34	10.3
8	19.2	.662	79	65	32	33	--
9	28.0	.862	88	69	28	41	--
10	21.8	.675	88	72	30	42	16.2
11	32.7	.940	95	70	28	42	14.1
12	11.5	.613	51	66	27	39	12.2
13	29.5	.848	95	68	30	38	12.5
14	27.1	.808	92	70	28	42	14.4
15	19.0	.710	73	70	28	42	14.4
16	34.8	.972	98	70	31	39	13.6

Table 7. Results of Swell Pressure Tests

Test no.	Initial moisture content	Final moisture content	Maximum vertical pressure	Vertical strain in/in.	Maximum lateral pressure	Lateral* strain in/in.
1	22.7	26.9	70	.002	81	.007
2	27.0	30.0	24	.001	19	.003
3	17.0	25.2	103	.007	113	.008
4	29.5	31.7	20	.001	10	.002
5	30.7	32.5	8	.000	8	.002
6	9.2	22.5	152	.003	117	.008
7	16.3	24.2	90	.002	77	.007
8	26.1	29.8	55	.001	62	.006
9	28.0	31.8	22	.001	30	.002
10	21.8	26.1	124	.006	105	.008
11	32.7	33.4	10	.001	11	.003
12	11.5	22.5	166	.004	149	.008
13	29.5	31.6	42	.001	32	.005
14	27.1	30.9	95	.003	91	.007
15	19.0	26.9	127	.004	117	.008
16	34.8	35.6	4	.000	4	.001

* Lateral strain was determined from a calibration curve prepared for the device.

into two groups. The first group, Tests 1 through 8, has a liquid limit range of 64 to 65 and plasticity index range of 34 to 38. The second group made up of Tests 9 through 16 have a liquid limit range of 69 to 72 and plasticity range of 38 to 42. The average plastic limit for both groups is equal.

The initial moisture content is shown plotted against maximum vertical pressure on Figure 4. From this plot it can be seen that for Group One a linear relation exists between these data. For Group Two considerable scatter was obtained and the relation between moisture content and vertical pressure was not linear.

It is interesting to note the large increase in pressure for a given moisture content encountered within the range of liquid limit and plasticity index, especially near the natural shrinkage limit. Extrapolation of the moisture content-pressure curves indicates the zero pressures will be developed for moisture contents ranging from 1.1 to 1.2 times the plastic limit. Undisturbed pieces of the soil reached moisture contents of from 38 to 45 per cent when submerged in water with no lateral or vertical restraint. This would indicate that swelling will occur for moisture contents greater than 1.2 times the plastic limit, but only a slight restraint is required to prevent such swelling. The swell pressure at the plastic limit varies from 2900 psf to 7000 psf. The low value corresponds to the low range of liquid limits and plasticity index.

A void ratio-log pressure plot of the vertical pressure data is shown on Figure 5. The break in the curves coincides very well with the preconsolidation load determined from the results of consolidation

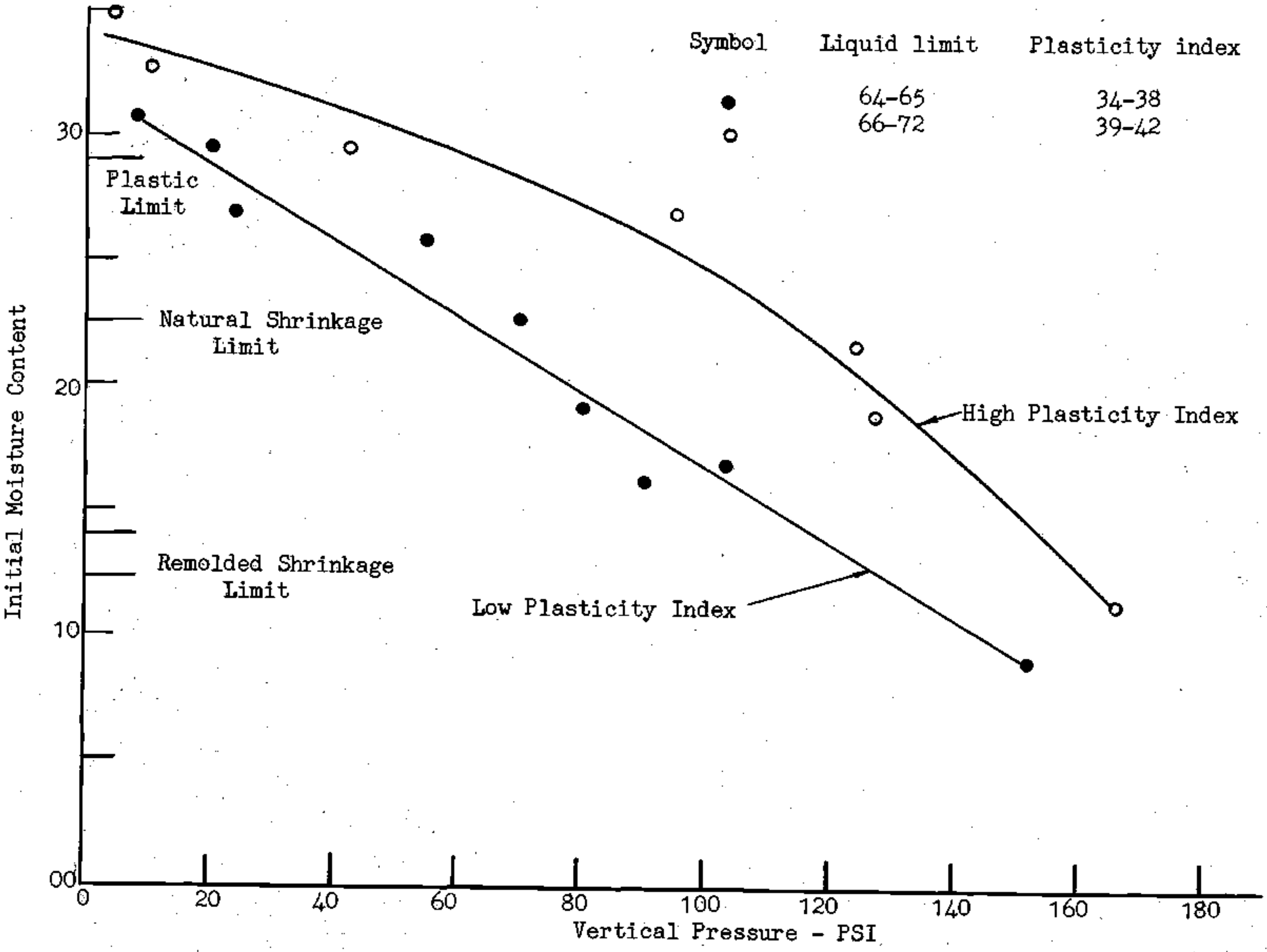


Figure 4. Vertical Pressure vs Initial Moisture Content for Volume Changes Less Than 2.3 Percent.

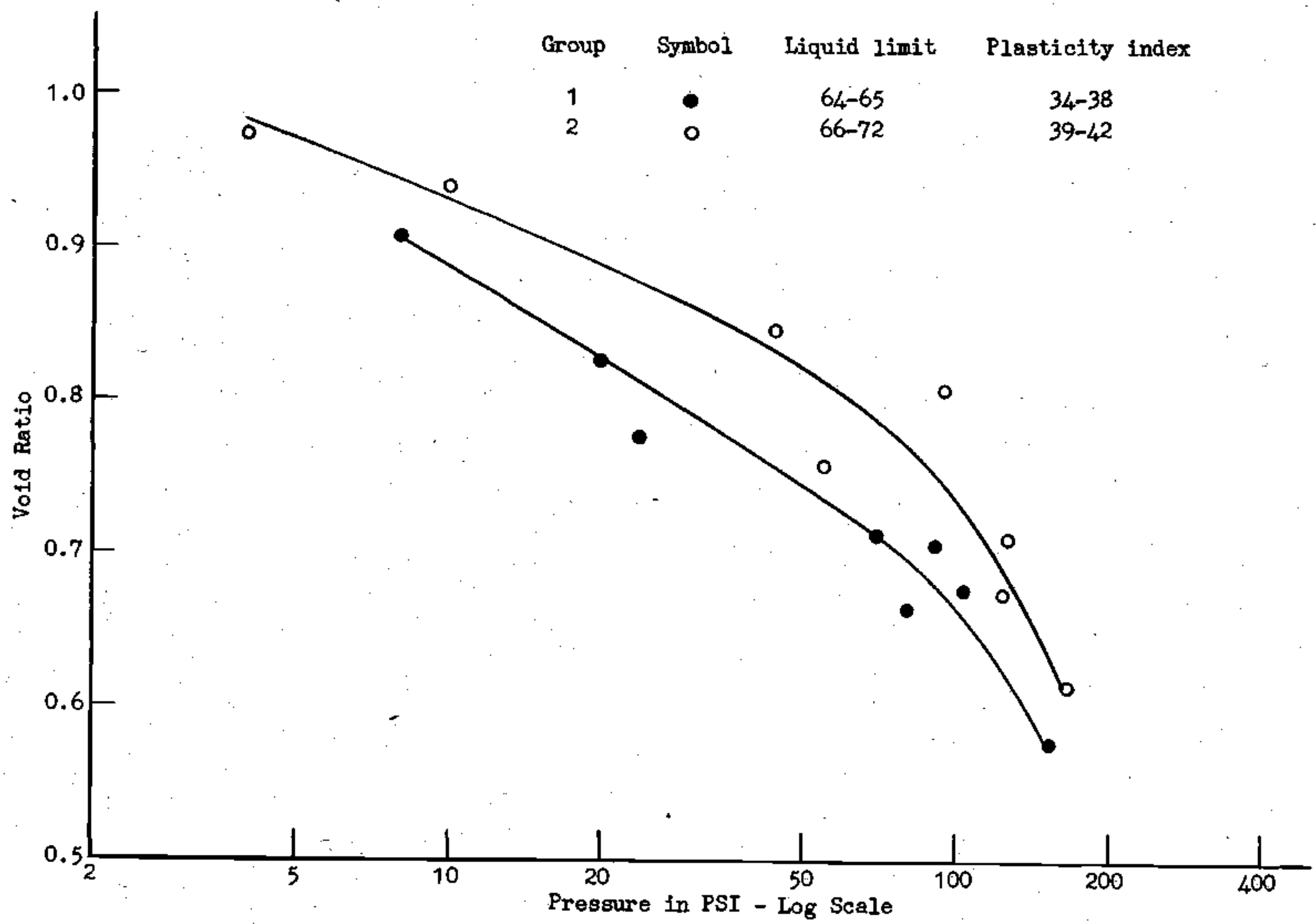


Figure 5. Void Ratio vs Log Vertical Swell Pressure.

tests. The consolidation tests were conducted in a 2.375 inch consolidometer with the consolidometer sealed to prevent loss of moisture. To compare the void ratio-swell pressure relation to the same relation obtained by consolidation tests, the average of two consolidation tests conducted on samples within the same liquid limit and plasticity index range have been superimposed on the void ratio-log of swell pressure plot of Group One in Figure 6. The initial void ratio was used in plotting the swell pressure data; if the void ratio of the swelled specimen is determined, these points will be slightly higher on the plot making up for only a small portion of the discrepancy between the data. This data demonstrates that the pressure required to consolidate a clay to a given void ratio is from four to five times greater than the swell pressure which will be produced by the same soil when restrained at that void ratio and allowed to freely imbibe water. The data is in general agreement with that of Clisby and Dawson (8).

Consolidation curves for samples having a liquid limit and plasticity index in the range of Group Two differ very little from the one shown of Figure 6, indicating that the discrepancy between the consolidation curve and the swell pressure curves is greatly reduced for the more plastic soil. Based on rather limited data, it is suggested that there is a force acting between the particles as a repulsive force resisting consolidation which does not enter into the swell mechanism of the soil. This could indicate that part of the consolidation is the result of a rearrangement of the soil particles. The force required to produce such a rearrangement would not occur in the swelling process. To substantiate this, it may be noted that full rebound does not occur

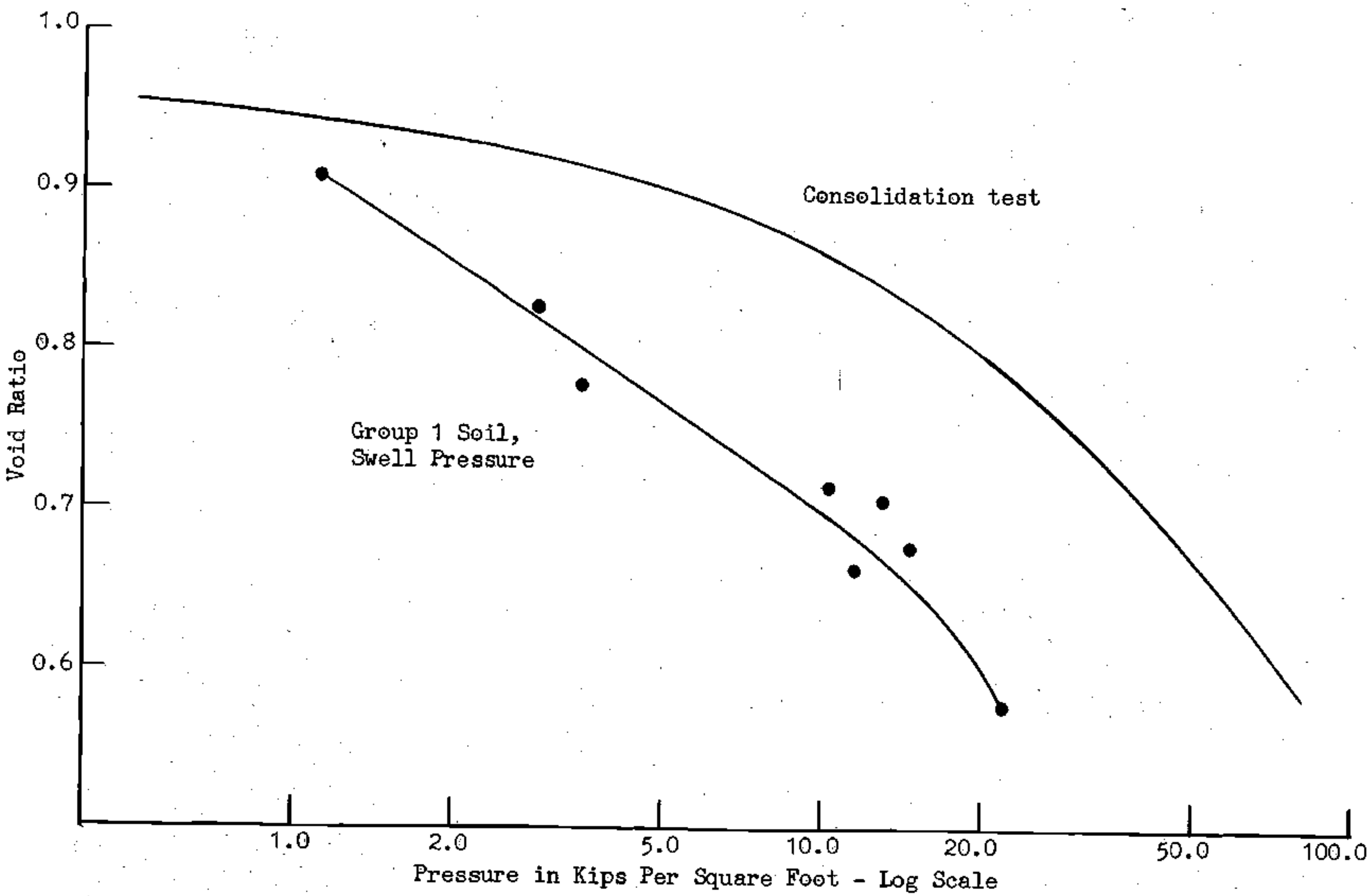


Figure 6. Compression - Swell Pressure Relation.

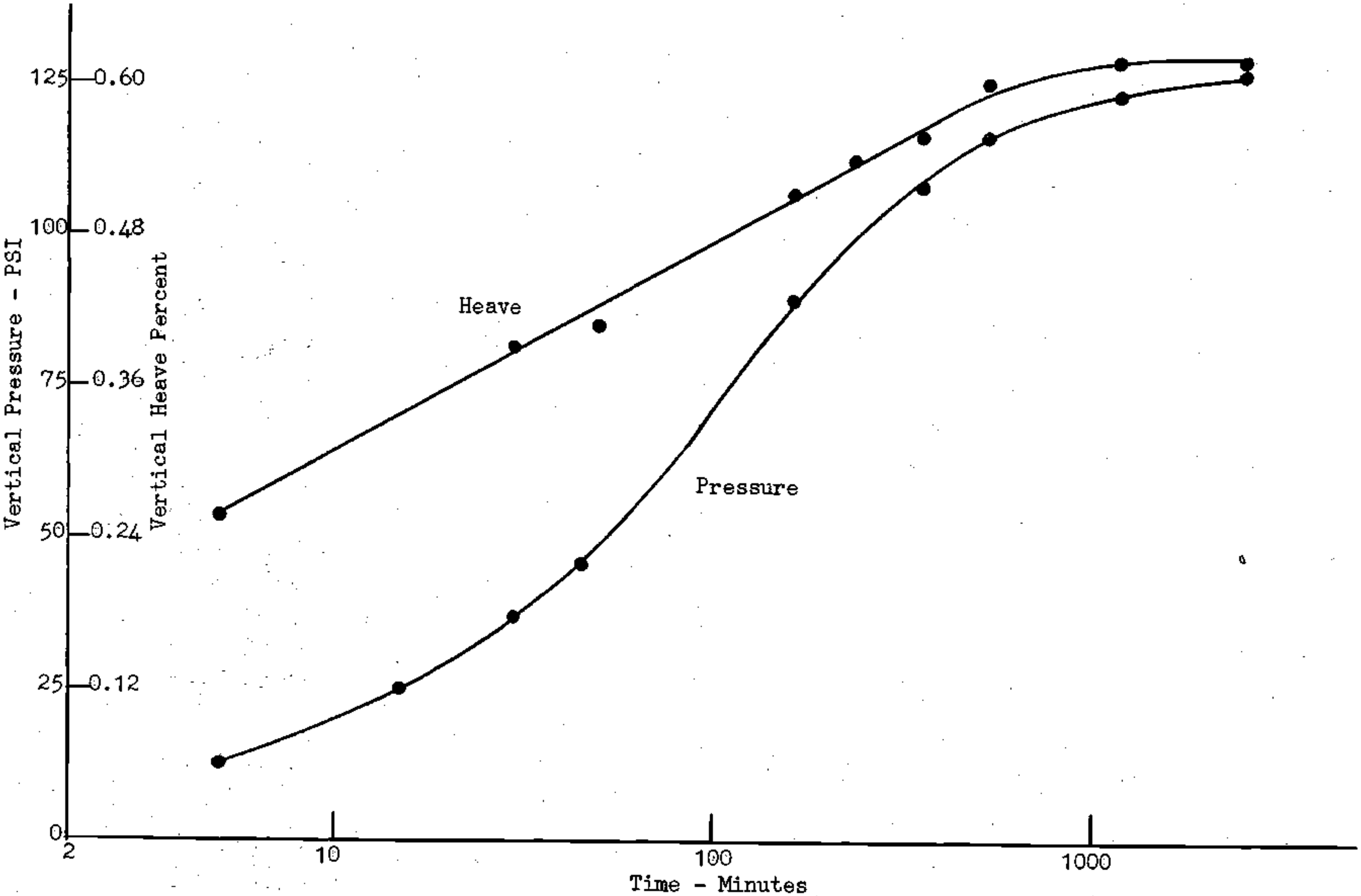


Figure 7. Typical Pressure-Time and Heave-Time Relations.

when the consolidation load is removed. It is also generally known that loading a soil may result in a change in particle orientation. This would indicate that while the consolidation specimens and the swell specimens have the same void ratio, it is possible that a different particle arrangement exists resulting in the different pressure.

The preceding discussion has been based on the assumption, for the most part, that the slight lateral swell which occurred in the swell pressure samples has not affected the vertical pressure. Obviously, it does; but then if the fit between the confining ring and the specimen in a consolidation test is considered, this effect is probably not too significant in the comparison made. The methods now used for placing the soil specimen in a consolidometer ring result in some clearance between the soil and the ring.

The Relation Between Vertical and Lateral Swelling Pressure

A plot of vertical swell pressure vs lateral swell pressure is shown by the solid dots in Figure 8. The solid line represents the hydrostatic stress condition. Vertical and lateral strains are indicated by circles, with the dotted line representing a condition where the lateral strain equals twice the vertical. As previously stated the pressure-strain relations are characteristics of the deformation of the device and, varying with the assemblage of the vertical measuring yoke, resulted in lateral strains equal to approximately twice the vertical.

It is interesting to note that despite the ratio of vertical to lateral strain of 0.5, the vertical and lateral swell pressures are approximately equal. A preliminary analysis of this data resulted in

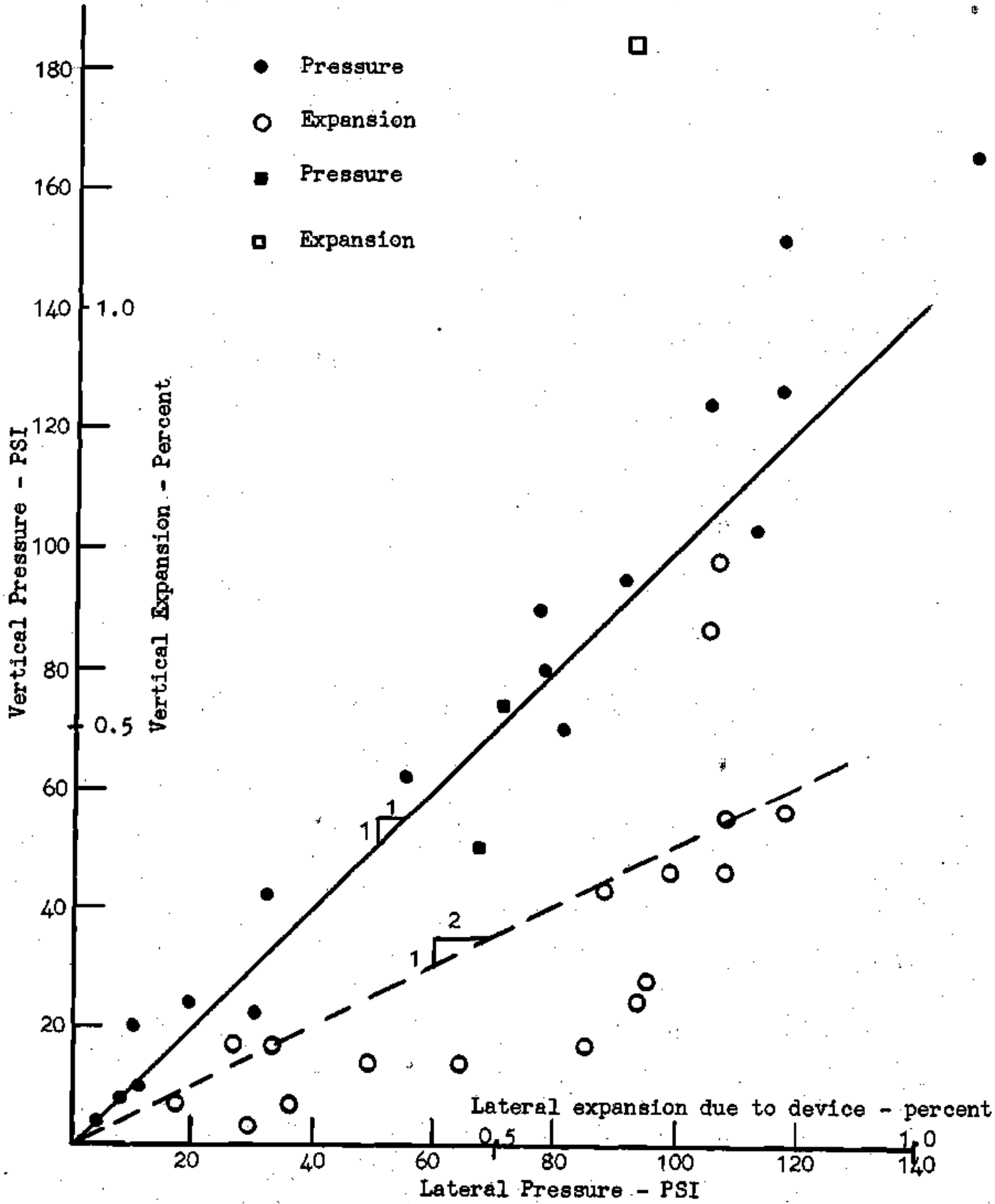


Figure 8. Vertical Pressure vs Lateral Pressure and Vertical Strain vs Lateral Strain

the misconception that the soil was capable of producing equal pressures while undergoing greater expansion laterally than vertically.

Shrinkage data (which will be discussed later) indicated vertical shrinkage deformations from 1.5 to 2.5 times the lateral deformations. Swell data from tests conducted in the consolidometers on specimens orientated with their bedding plane horizontal and specimens trimmed from samples rotated 90 degrees about a horizontal axis indicated that the clay was capable of swelling in a direction normal to the bedding plane of the soil much more than in a direction parallel to it. The pressure and strain ratio data as interpreted is not consistent with the latter data. Several tests were conducted in which the specimen was allowed to expand freely in a vertical direction for 1.0 and 2.5 per cent and then restrained. This resulted in vertical expansion or strains of approximately two and four times greater than the lateral strain. The pressure data is plotted as solid squares and the strain as hollow squares on Figure 8. The strain plot for the large ratio fell off the graph and is not shown. The pressure point for the smaller ratio fell almost on the hydrostatic line while the other fell below as would be expected.

These data along with the previously mentioned inconsistencies suggest that the swelling process acts somewhat like a hydrostatic pressure so long as the ratio of strains is less than two and results in a plastic flow of the soil so that it adapts itself, within limitations, to the shape of the confining conditions. At ratios of strain greater than two, this ceases.

In any test of this type, the effect of the boundry conditions should be considered. If no volume change is allowed to occur, there should be no effect. This is based on the assumption that the swell pressure build-up results from a uniform release of attractive particle forces on planes parallel to the top and bottom confining stones. The release starts at the contact surface and progresses toward the center of the specimen. No restraint is offered by the rubber membrane and confining fluid, under these conditions.

Under the true conditions, (deformation occurring), the lateral restraint afforded by the porous stones would result in the measured lateral pressure being less than that which would occur if no restraint existed. A thicker specimen would reduce this difference; but considering the small deformation which occurred and the restraining force being limited to the shear strength of the specimen, it was felt that this effect is negligible.

<u>Initial Moisture Content vs Gain in Moisture Content in Swell Pressure Tests</u>

The initial moisture content is shown plotted against the gain in moisture content for the sixteen swell pressure tests in Figure 9. There is a definite break in the curve drawn through these points between 20 and 25 per cent initial moisture content, corresponding to the natural shrinkage limit. As would be expected, the gain in moisture content is dependent on the degree of saturation and the amount of swell allowed. There is no indication that the gain is related to the index properties of the soil.

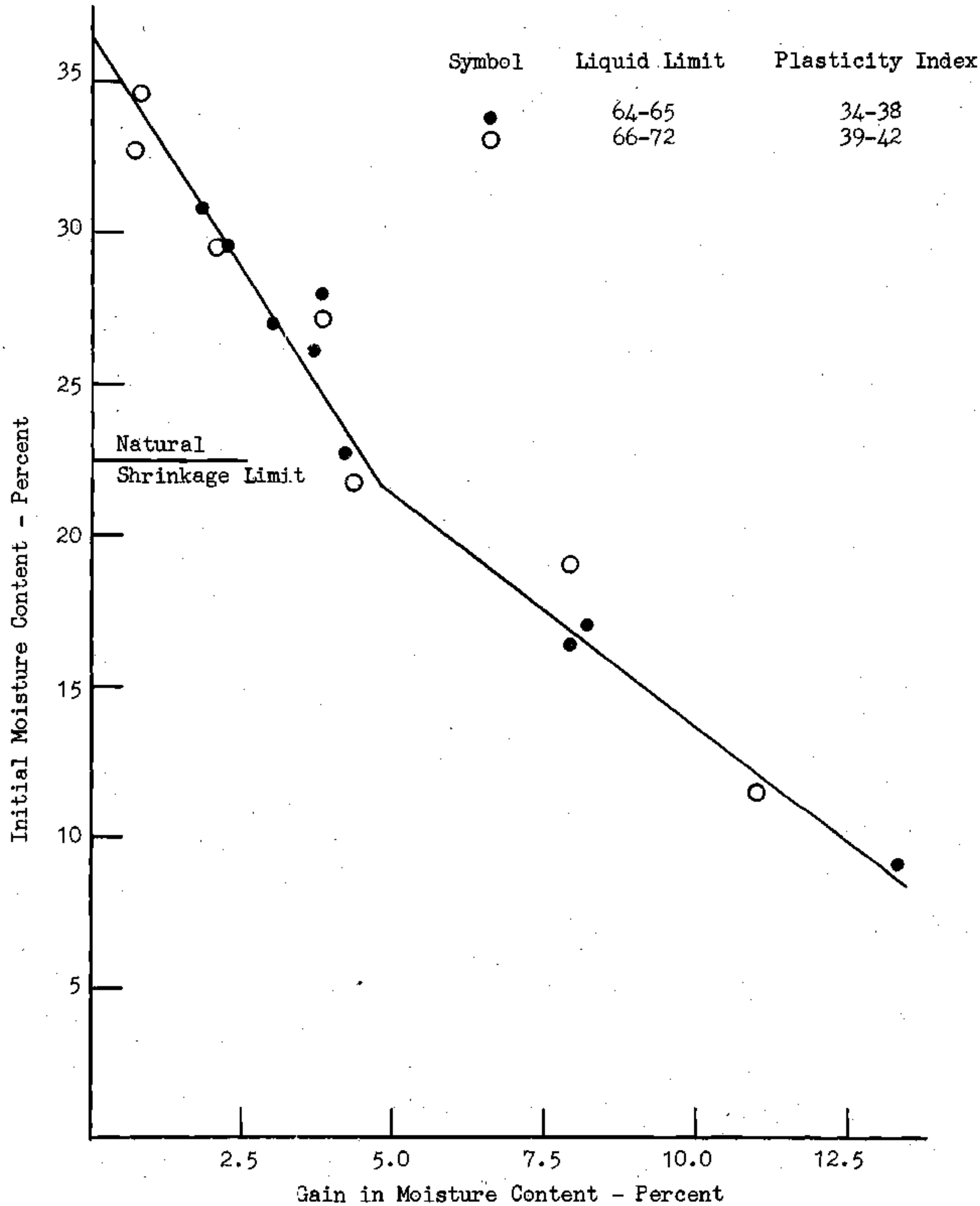


Figure 9. Initial Moisture Content vs Gain in Moisture Content.

The Relation of Void Ratio to Moisture Content

The moisture content of the Yazoo Clay as received in the laboratory varied between 30 and 35 per cent. To determine the swell characteristics at lower moisture contents, it was necessary to dry some of the samples as described in Chapter VI. Prior to testing, the moisture content (initial moisture content in Table 6) and void ratio of each specimen was determined. This data is presented as Void Ratio vs Moisture Content graph in Figure 10. The values for zero moisture were obtained from the oven-dried specimens after completion of the swell tests. Data from all specimens prepared for the various tests are shown on Figure 11. It was not feasible to determine the void ratio for zero moisture for all of these specimens, but sufficient values were obtained to show the range of void ratios that could be expected for the oven-dried condition. The void ratio for zero moisture content was determined only for those specimens which did not crack or in which only slight cracking occurred.

For moisture contents above 23 per cent the degree of saturation falls between 90 and 100 per cent with the higher saturations corresponding to higher moisture content.

The data indicates that as desiccation occurs the degree of saturation decreases and that a linear relation exists between void ratio and moisture content for moisture contents above 20 to 25 per cent. Below this, there is considerable scatter in the data, but a definite pattern is formed indicating a change in resistance to shrinkage. This point represents the Natural Shrinkage Limit of the soil. A decrease in moisture content below the Natural Shrinkage Limit is accompanied

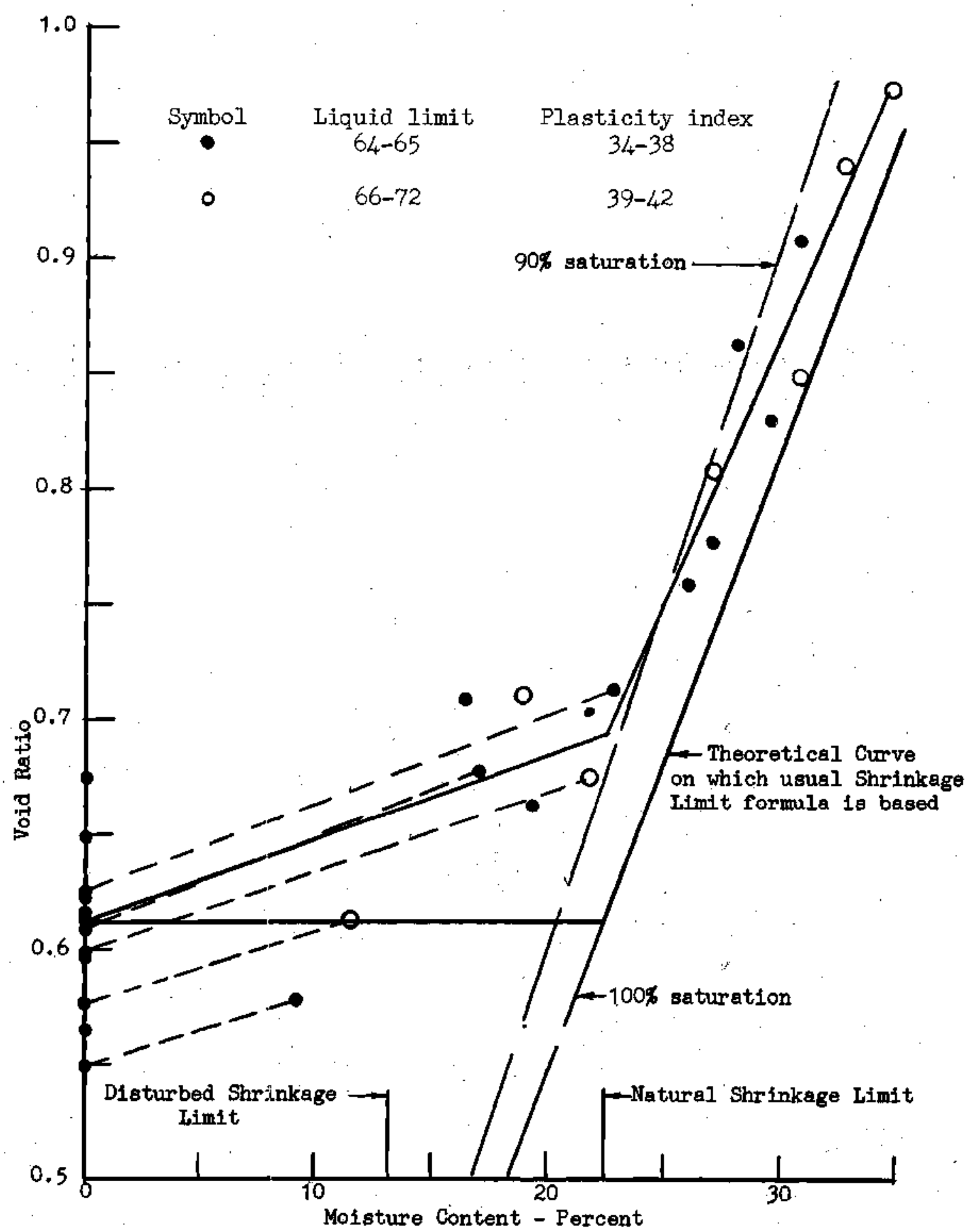


Figure 10. Void Ratio vs Moisture Content.

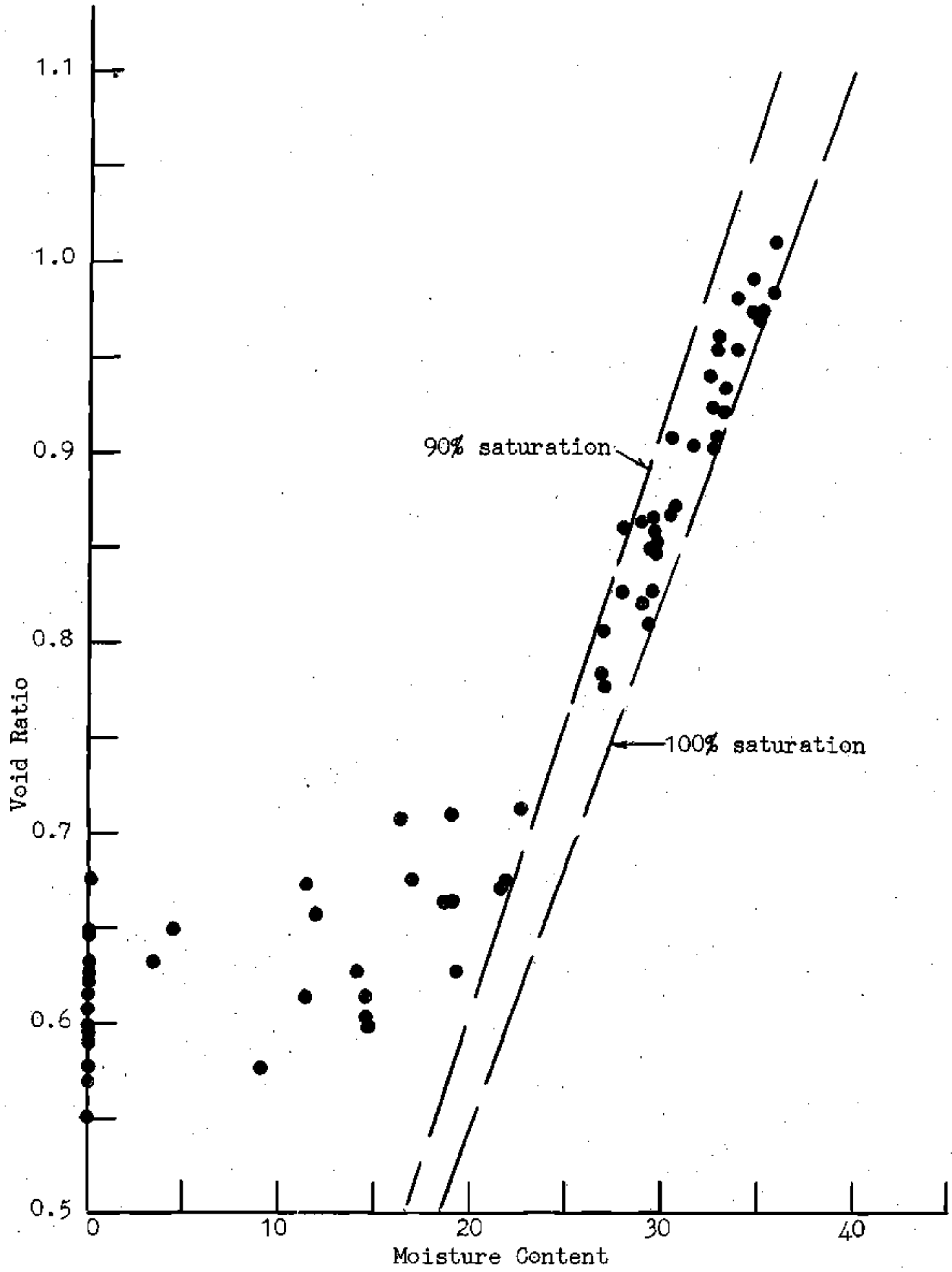


Figure 11. Void Ratio vs Moisture Content for All Specimens.

by very small change in void ratio. Nothing in these data suggests that at any time is there desiccation without some change in void ratio. This is not the case for clays of low plasticity or for silts.

The void ratio determined from the shrinkage limit tests shows a range of from 0.442 to 0.582 with an average of 0.490, a value considerably below the void ratio of the undisturbed soil when oven-dried. The above value does not agree with the void ratio value computed from the shrinkage limit assuming 100 per cent saturation. An average value of 0.354 is obtained in this manner. The shrinkage limit as computed from the higher void ratio value is approximately 18 per cent. The discrepancy is due to the standard test procedure (ASTM D427-39) for determining shrinkage limit which neglects the possibility of air being trapped in the remolded wet soil pat. All of this entrapped air, by this procedure, becomes part of the volume of voids of the pat but does not enter in the calculations.

While the above discrepancy is primarily of academic interest only, it does indicate a possible source of trouble in relating the standard shrinkage limit from remolded soils to the expansive potential of soil.

The difference in the undisturbed shrinkage limit and the shrinkage limit determined in the standard test on remolded soils serves to point out the effect which the natural structure of soil has on the shrinkage characteristics. It is interesting to note that the linear shrinkage data shows greater shrinkage normal to the bedding plane than parallel to it. This indicates, to some degree, a parallel orientation of soil particles in the undisturbed soil. The remolded soil shows uniform shrinkage indicating random orientation. It is generally agreed that parallel orientation is more inducive to obtaining denser soil conditions. The void ratio data cited above for oven-dried undisturbed and remolded conditions do not agree with this theory. These data suggest that the remolding not only changes the particle orientation but destroys some source of repulsive force between the particles.

It is impossible to say what fluctuation of moisture contents will occur under field conditions; but in the range above about 23 per cent, any decrease will most certainly result in considerable volume decrease as indicated by the data in Figure 11.

Anisotropic Characteristics as Noted by Shrinkage

Early in this study it was noted that the shrinkage of undisturbed geometric shapes of the Yazoo Clay was not uniform. Measurements indicated that the shrinkage normal to the bedding plane of the soil was greater than the lateral shrinkage. To substantiate these observations several series of simple shrinkage tests were conducted on undisturbed and remolded specimens, or data were obtained from specimens prepared for other tests. A summary of this data is given in Table 8.

These data indicate that, on the average, the vertical shrinkage from near the plastic limit to an air-dried condition is 1.95 times the lateral shrinkage. For remolded specimens the ratio is 1.13. The variation in this relation is quite large and may be attributed to both natural variations in the soil and disturbance in preparing the specimens. The lateral shrinkage was obtained by averaging two measurements made at right angles. The difference in these two values was

Table 8. Shrinkage Measurement Data

Specimen no.	Shape	Soil condition	Initial moisture content percent w_i	Vertical unit shrinkage percent ϵ_v	Lateral* unit shrinkage percent ϵ_L	ϵ_v / ϵ_L
1	prism	undisturbed	28.0	7.4	3.2	2.31
2	prism	undisturbed	26.7	4.7	2.3	2.35
3	cylinder	undisturbed	34.2	9.0	5.7	1.58
4	prism	undisturbed	32.0	8.9	4.8	1.85
5	prism	undisturbed	32.8	9.6	4.7	2.04
6	prism	undisturbed	34.4	10.0	5.7	1.75
7	prism	undisturbed	33.2	9.8	5.4	1.81
8	prism	undisturbed	33.3	9.3	5.3	1.76
9	cylinder	remolded	33.7	8.6	8.3	1.04
10	cylinder	remolded	33.7	8.1	7.3	1.11
11	cylinder	remolded	33.7	9.6	7.8	1.23
12	prism	undisturbed	36.1	9.1	6.0	1.48
13	prism	undisturbed	--	10.2	4.3	2.37
14	prism	undisturbed	--	9.6	4.0	2.40
15	cylinder	undisturbed	--	3.6	2.0	1.78
16	cylinder	undisturbed	--	3.1	2.4	1.26
17	prism	undisturbed	--	8.7	3.4	2.56
18	prism	undisturbed	--	7.3	3.8	1.92
		Average undisturbed.				1.95
		Average remolded				1.13

*Average of two measurements made at right angles.

insignificant. These findings indicate an orientation of the flake-shaped clay particles so that the flat surfaces tend to be parallel to the bedding plane.

Lambe (23) indicates the ratio of width to thickness for montmorillinite clay particles is approximately 100 and the ratio of length to width is about one. If perfect orientation of this material was obtained in a unit mass of soil, the total height of voids normal to the particle forces would be about 100 times the length of the lateral voids. It is assumed that the face to face and the edge to edge spacing is equal. If the change in spacing due to shrinkage of both the face to face and edge to edge voids is equal, it is possible to have a ratio of normal to lateral shrinkage in the neighborhood of 100. From this standpoint the maximum value of 2.56 listed in Table 8 appears insignificant, but when the total effect on a mass of soil is considered, the slightest tendency toward orientation may be important.

In the deposition of marine clays there is a slight tendency toward orientation of particles parallel to the bedding plane, but in general, a haphazard arrangement is predominant. As the overburden pressure increases the particles tend to assume an orientated position parallel to a horizontal plane.

The ratio of vertical to lateral shrinkage for the remolded soil of about 1 indicates that the kneading action used in preparing these specimens results in a random arrangement. The fact that the average ratio for these specimens is greater than one is attributed to a very slight orientation due to the static force used in forming the remolded specimens.

The specimens for tests 3 through 11 were obtained from one sample. The initial moisture content of these specimens was very uniform indicating, if saturated, uniformity of void ratio. If the volumetric shrinkage ($\varepsilon x + \varepsilon y + \varepsilon z$) is determined, it is found that an average value of 0.020 is obtained for the undisturbed specimen and 0.024 for the remolded, indicating an increase in volumetric shrinkage of 20 per cent for the remolded soil.

In the air-dried condition the remolded or random oriented soil is in a more dense state than the undisturbed oriented soil. This does not agree with results from the sedimentation of dispersed and flocculated sediment as reported by Lambe (23).

Linear Shrinkage Tests

The linear shrinkage tests were set up to obtain values of the vertical movement under a nominal load. The setup is an over simplification of a footing resting on the surface of the soil. The data for these tests are given in Table 9. Oddly enough the undisturbed specimens under the 100 psf vertical load showed less vertical shrinkage than the other test specimens under no load. These specimens, one inch thick and 2.8 inches in diameter showed more tendency to crack than the prisms (approximately 1.5 inches on a side) or the cylinders one inch high and 1.4 inches in diameter. This difference is probably the result of the complete coverage of the base and partial coverage of the top of these specimens. No cracks occurred in the remolded specimens.

It was hoped that this test or a similar one measuring the shrinkage from approximately the plastic limit to the air-dried state

Table 9. Linear Shrinkage Test Results

Specimen no.	Soil condition	Initial moisture content percent w_i	Vertical unit shrinkage percent ϵ_v	Lateral unit shrinkage percent ϵ_L	ϵ_v/ϵ_L
1	undisturbed	37.1	8.7	5.0	1.74
2	undisturbed	35.5	5.8	4.8	1.21
3	remolded	35.9	8.4	8.5	.99
4	remolded	36.2	10.7	9.0	1.19
5	undisturbed	34.3	7.1	4.5	1.58
6	undisturbed	32.7	5.5	4.8	1.15
7	remolded	32.9	7.7	7.2	1.07
8	remolded	32.9	8.2	7.9	1.04
	Average undisturbed.		6.8	4.8	1.42
	Average remolded		8.7	8.1	1.07

would provide a way of numerically expressing the expansive characteristics of a soil. Although only a few tests were conducted and the data are not extensive, several things were learned. The data indicate that any use of shrinkage of remolded samples can be highly misleading. It is also apparent that orientation of undisturbed samples as they existed in the field must be known if linear tests are used. Volumetric shrinkage tests on undisturbed specimens can also be misleading if there is a difference in vertical and lateral shrinkage.

The test is simple to run, and by controlling the rate of drying, cracking can be controlled. By obtaining weight as well as shrinkage readings periodically the linear shrinkage from the plastic limit to the air-dried condition can be determined. In the tests reported here, the initial moisture content of the specimens was approximately equal to the plastic limit, making it unnecessary to obtain weight data. The shrinkage range was taken as the difference between the initial and final dial readings. It is impossible from so few tests on soil from one location to determine if this shrinkage index is truly characteristic of the expansive properties of a soil. A comprehensive testing program is required for this purpose.

The test also provides a limit on the range through which a soil may shrink when subjected to long drought. If the initial moisture content is at the no swell point, slightly above the plastic limit, then a maximum range of movement, either up or down is obtained.

The Effect of Anisotropy on Swelling and Consolidation

The results of the shrinkage tests indicate that the Yazoo Clay is definitely anisotropic. To study the effect of the anisotropy of the specimens on the swelling mechanism, four series of tests were conducted. Half of the specimens in each series was loaded normally and half was loaded parallel to the bedding plane of the soil. The results of these tests are shown as void ratio vs log pressure plots in Figures 12, 13, 14 and 15. Numerical data giving the change in void ratio between significant points are given in Table 10.

In general the data show that under a nominal load the Yazoo Clay will swell more vertically than horizontally. The ratio of swell for each series is given in Table 11. Neglecting the data for series three because of the modified sequence in conducting that series, the average ratio of vertical to horizontal swell is 2.33 with good agreement between series.

Close agreement in these tests is also obtained for ratio of void ratio change between 500 psf and maximum load. This ratio for series three indicates that the orientation has little effect and that the difference exhibited by the other test series is due to difference in swelling. A careful study of the initial void ratios indicates that for series two, the specimen load parallel to the plane of preferred orientation had a higher initial void ratio but experienced less consolidation under loading. Based on the data obtained for the double oedometer tests, the variation of the data of series three is more likely attributable to effects produced by not conducting the test in an immersed condition.

The void ratio changes due to rebound are very consistent. The odd shape of the rebound curve for series three is probably due to the

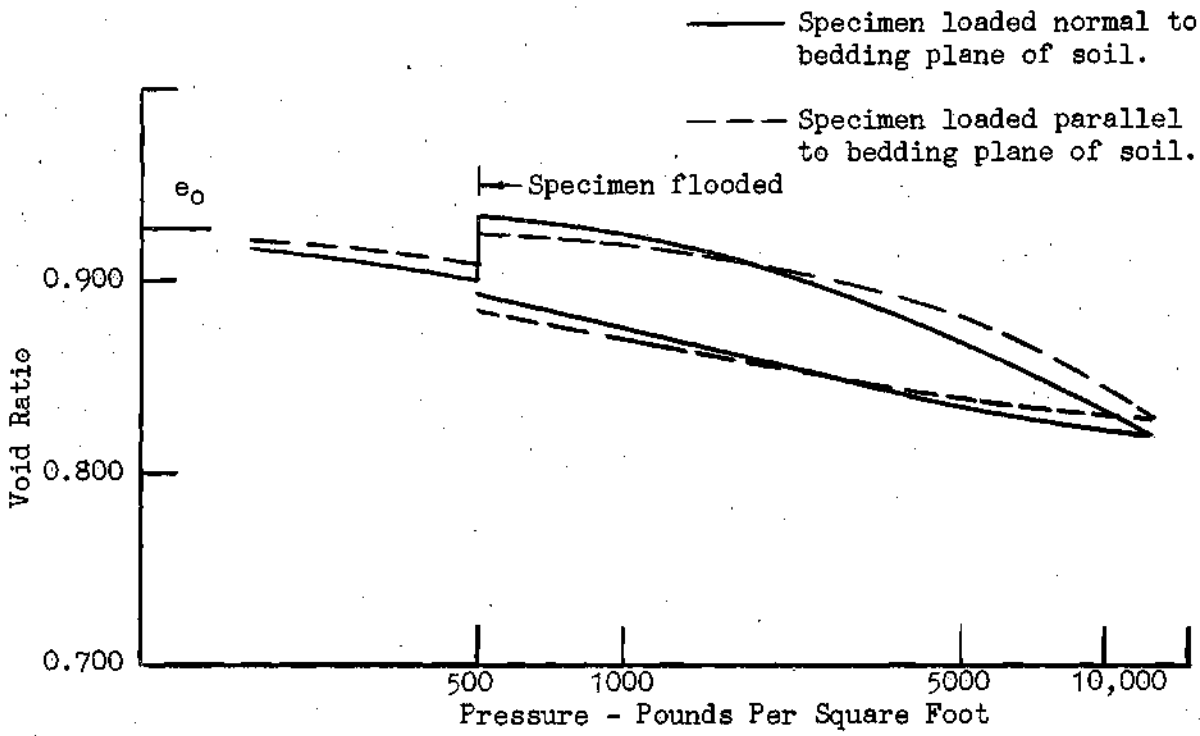


Figure 12. Void Ratio vs Log Pressure, Test Series No. 1.

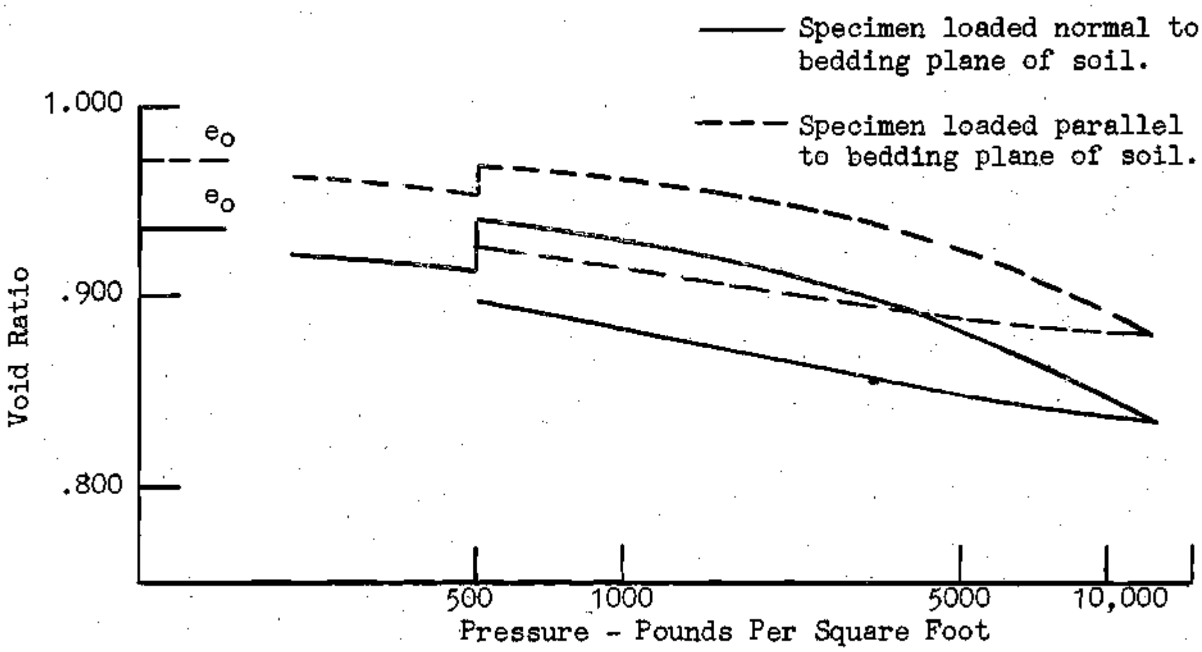


Figure 13. Void Ratio vs Log Pressure, Test Series No. 2.

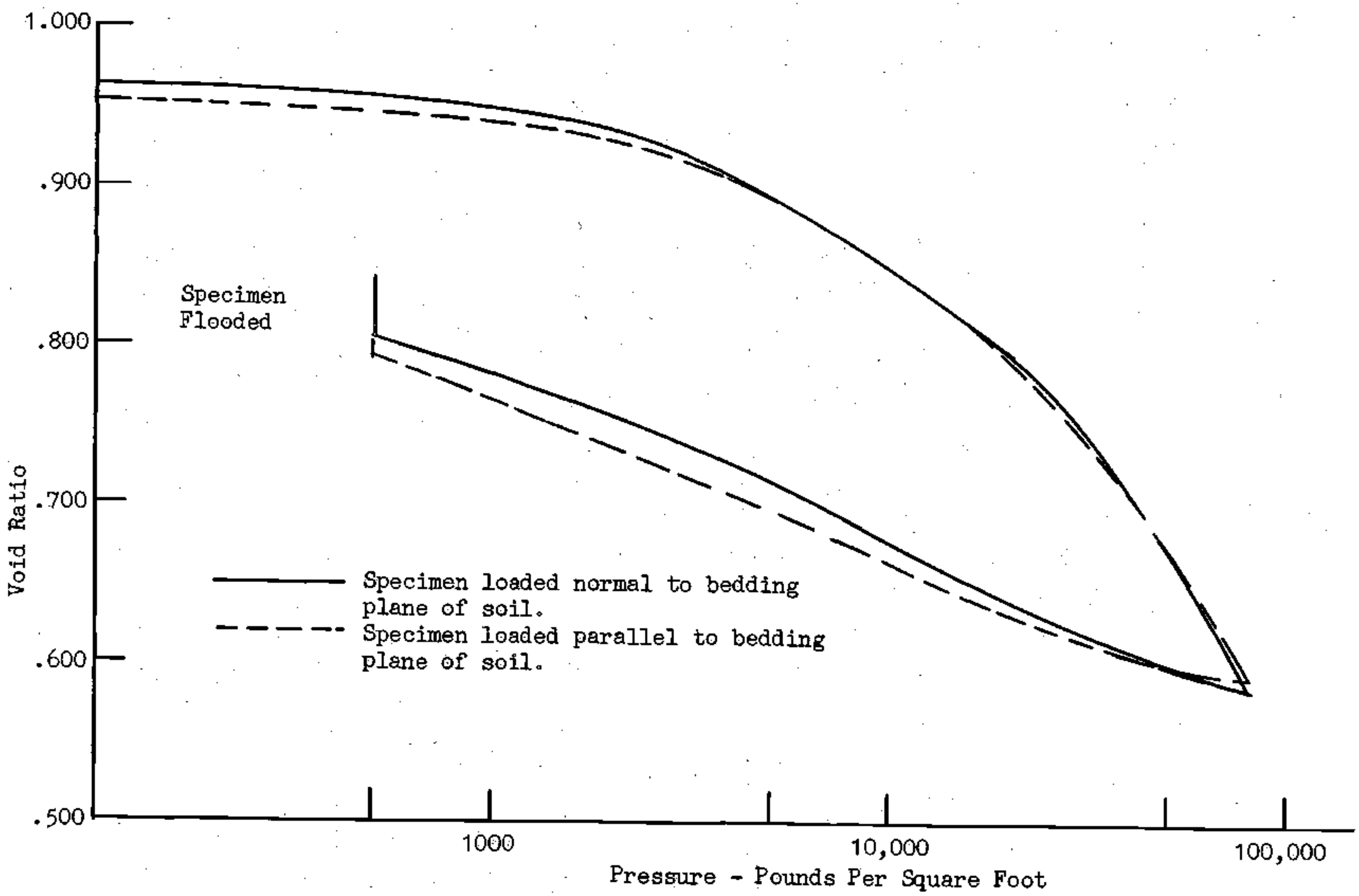


Figure 14. Void Ratio vs Log Pressure, Test Series No. 3.

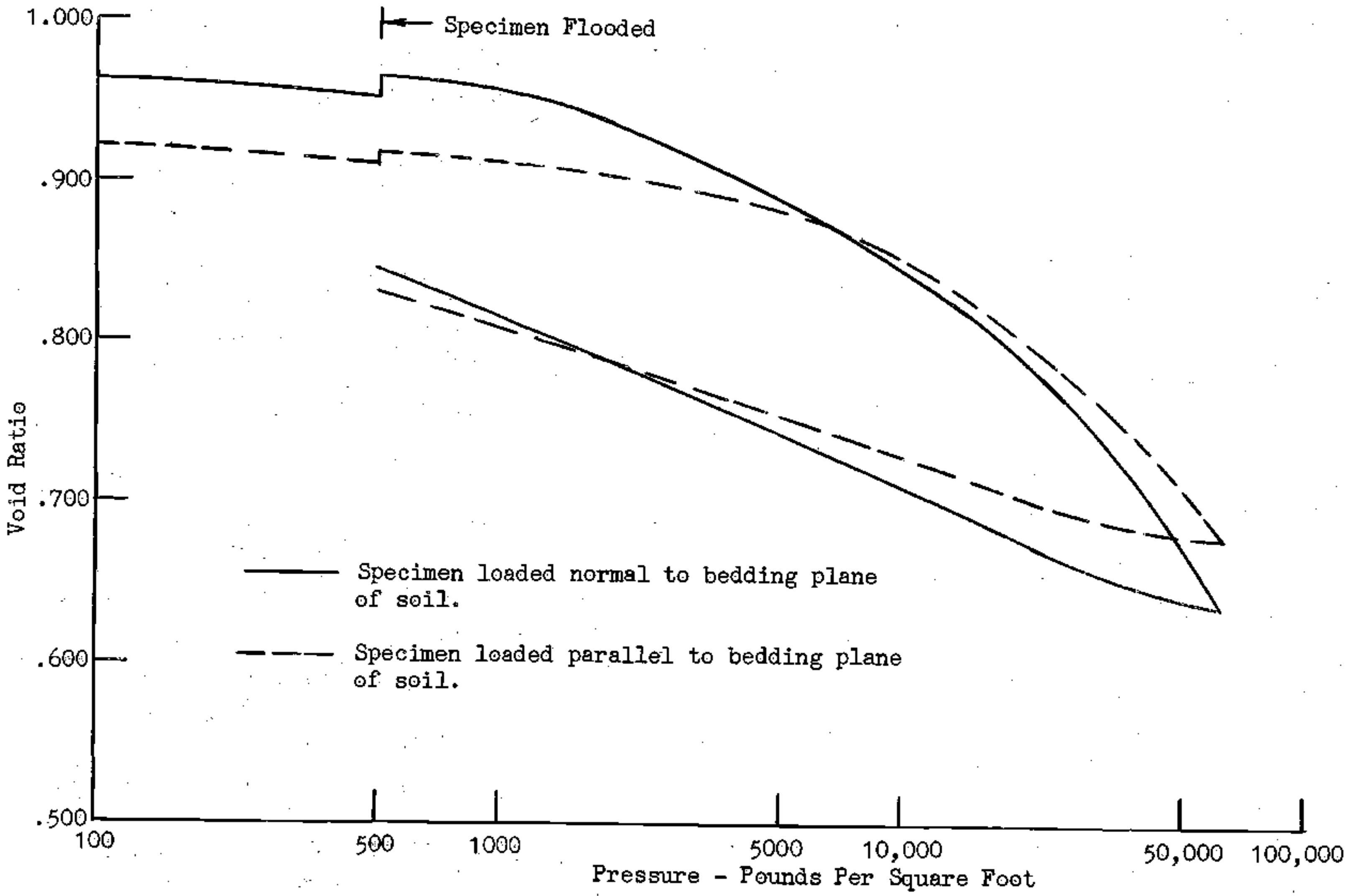


Figure 15. Void Ratio vs Log Pressure, Test Series No. 4

Table 10. Results of Consolidation Tests on Specimens Loaded Normal and Parallel to Bedding Plane

Test series no.	Consolidometer diameter	No. specimen	Orientation	Δe_o	Δe_1*	Δe_2**	Δe_3***	Δe_4****
1	1.4	4	Normal	0.928	0.029	0.033	0.113	0.074
	1.4	4	Parallel	0.928	0.017	0.012	0.095	0.054
2	1.4	3	Normal	0.936	0.023	0.026	0.104	0.062
	1.4	3	Parallel	0.971	0.019	0.014	0.087	0.043
3‡	2.4	2	Normal	0.963	0.008	0.038	0.369	0.219
	2.4	2	Parallel	0.952	0.008	0.011	0.353	0.200
4	2.4	2	Normal	0.961	0.010	0.012	0.325	0.207
	2.4	2	Parallel	0.921	0.011	0.005	0.238	0.151

* Change in void ratio from seating load to 500 psf.
** Change in void ratio during flooding.
*** Change in void ratio from 500 psf to maximum load.
**** Change in void ratio during rebound to 500 psf.
‡ Specimens were flooded after rebound.

Table 11. Ratios of Change in Void Ratio

Test series no.	$\frac{\Delta e_{1v}}{\Delta e_{1L}}$	$\frac{\Delta e_{2v}}{\Delta e_{2L}}$	$\frac{\Delta e_{3v}}{\Delta e_{3L}}$	$\frac{\Delta e_{4v}}{\Delta e_{4L}}$
1	1.7	2.75	1.19	1.37
2	1.2	1.86	1.20	1.44
3*	1.0	3.45	1.05	1.10
4	0.9	2.40	1.37	1.37

* Specimens were flooded after loading and rebound.

build-up of negative pore pressure. If the change in void ratio for swelling occurring after flooding is added to the rebound change, a ratio of void ratio change of 1.22 is obtained, a value more in line with the other data. The results of these tests are about what was expected in view of the data obtained from the shrinkage measurements. It is obvious that this soil is not isotropic and that the magnitude of swelling is dependent to some degree on the orientation of the clay particles.

Height and diameter measurements were made on the specimens from series three after removal from the consolidometer and after oven drying. The specimens had been loaded to a pressure of 80,000 psf. Measurements on cubes of soil from the same sample give a ratio of 2.24 for vertical to lateral shrinkage. The ratio for the consolidation specimen loaded normal to the plane of orientation was 3.34. For the specimen loaded parallel to the orientation plane, a value of 0.69 was obtained for the ratio of shrinkage normal to the original plane of orientation, to the shrinkage parallel to the axis of loading. The unit shrinkage in the third direction did not appear to be affected. This data substantiates the belief that loading can result in a reorientation of the clay minerals.

Long Time Swell Test Results

The long time swell tests are a continuation of the swell pressure tests and the data for the specimens used are given in Table 6 as tests nos. 7 and 8.

Tests nos. 7 and 8 were allowed to run for 43 and 94 days respectively. The first was terminated because of difficulties with leakage in the cell and the second, because of time. In both instances the pressures were still increasing with time at the termination.

The pressure-time and pressure-strain data for these tests are shown in Figures 16 and 17. As previously discussed, the strain curves are characteristic for the pressure cell. There is no evidence that the shape of the strain curves affected the general shape of the pressure curves.

The swell pressure build-up in these specimens appears to have been made up of two distinct processes. Initially, there was a very rapid pressure increase, the rate of which was dependent on the ability of water to enter the soil. The second process is much slower and may be called secondary swell. A study of the swell pressure test results indicated that several of these specimens showed a tendency for secondary swell, others did not. There was no correlation between this tendency and the index properties of the samples although there may be some relation with initial moisture content. The dryer specimens are more likely to show long time swell. If this is true, the long time swell properties may be effected by the length of time the specimen remains at the low moisture content prior to swelling.

There are insufficient data to try to assign the various swelling mechanisms to one or the other of the time phases, but it would appear logical that the long time swell characteristics are the result of a physico-chemical phenomena occurring in the innermost adsorbed water layers. It is generally agreed that this water possesses properties unlike free water due to the extreme forces exerted on the water molecules

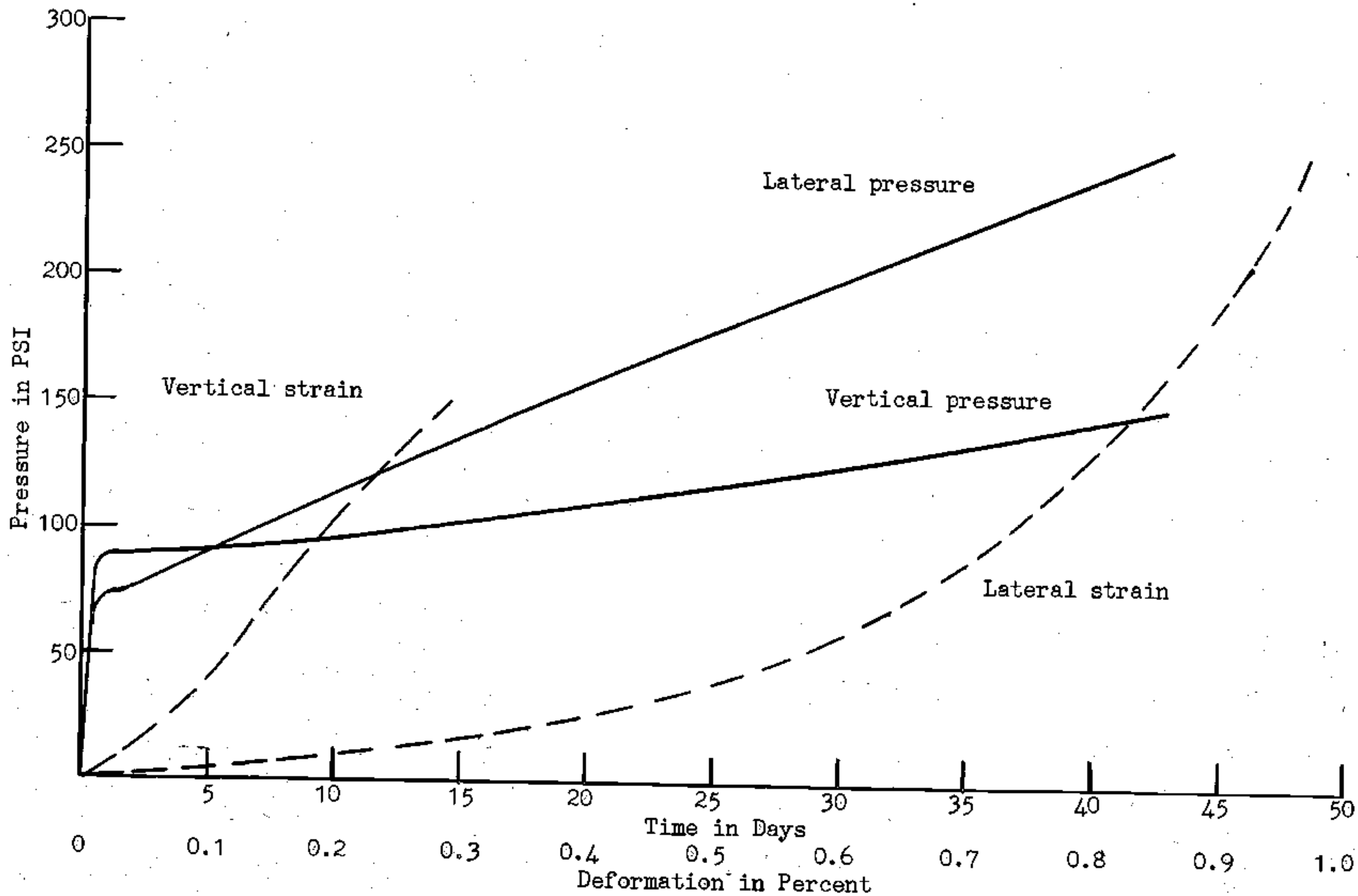


Figure 16. Pressure vs Time and Strain

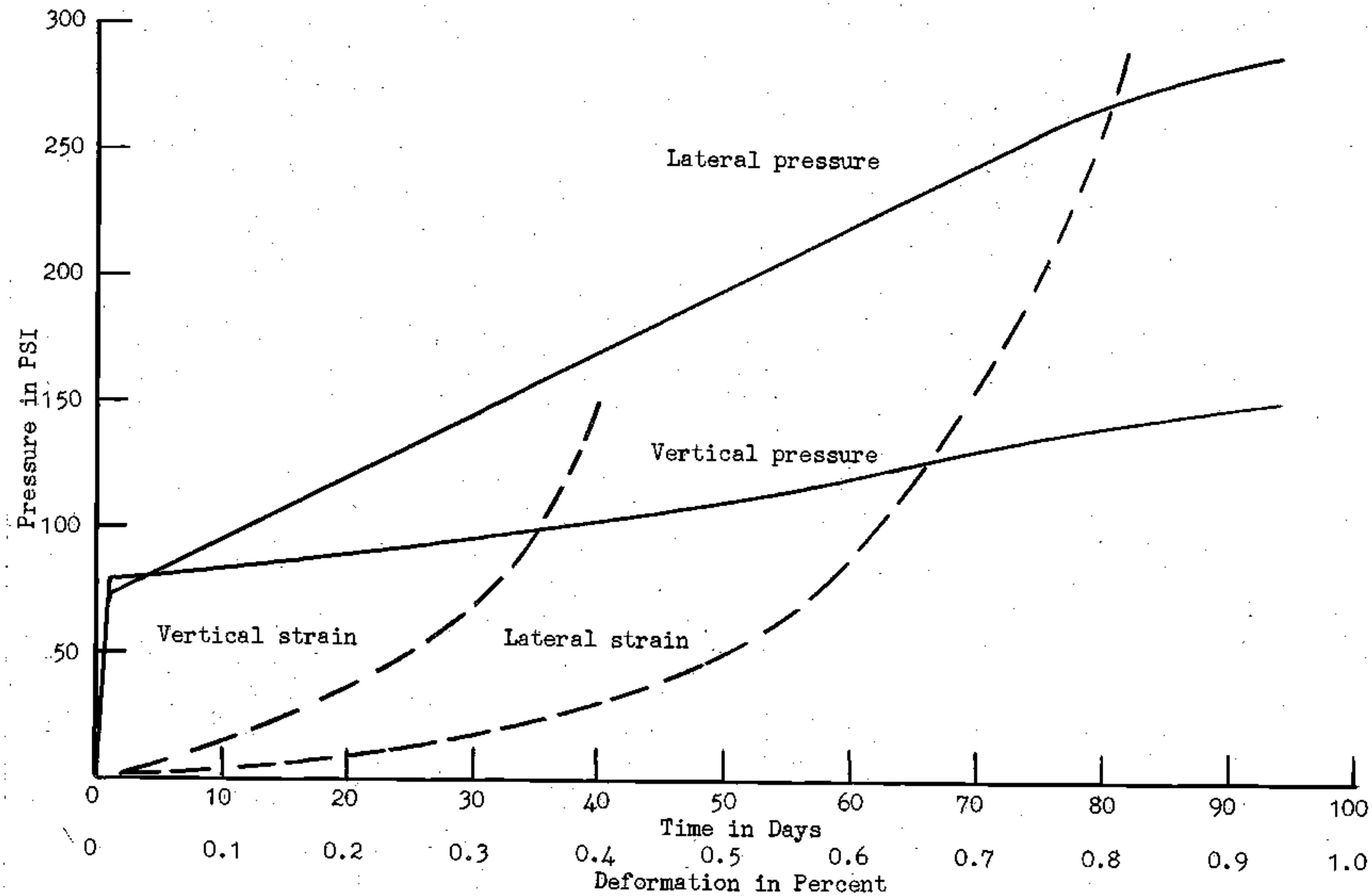


Figure 17. Pressure vs Time and Strain

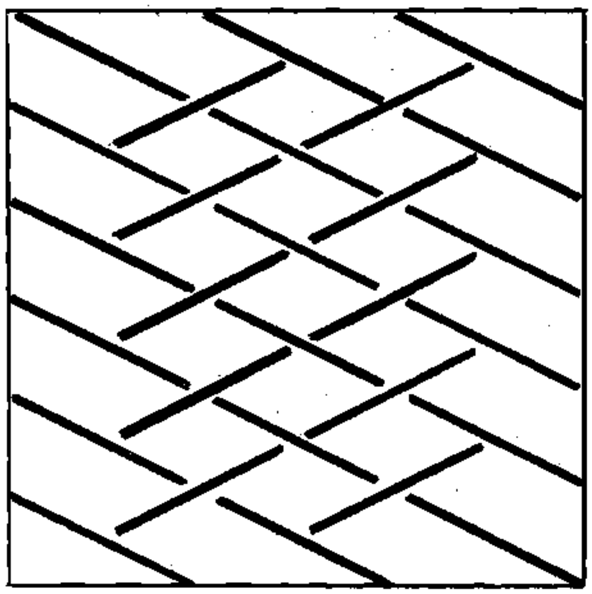
near the clay mineral surface. The time needed for hydration of adsorbed ions in this water may be quite long.

Of equal interest and just as baffling is the relative magnitude of the long time swell pressure. In both tests the lateral pressure increased at a faster rate during the secondary swell phase and was at the end of the tests 1.7 and 1.9 times the vertical pressure. A check of the pressure-measuring equipment was made to determine if the high lateral pressure caused the o-ring to bind the vertical loading ram. The results of this check indicate that the effect is negligible when compared to the difference in the two pressures. Every effort was made to determine if the difference was due to some malfunction of the cell, but nothing was found to indicate that the cell was not functioning properly.

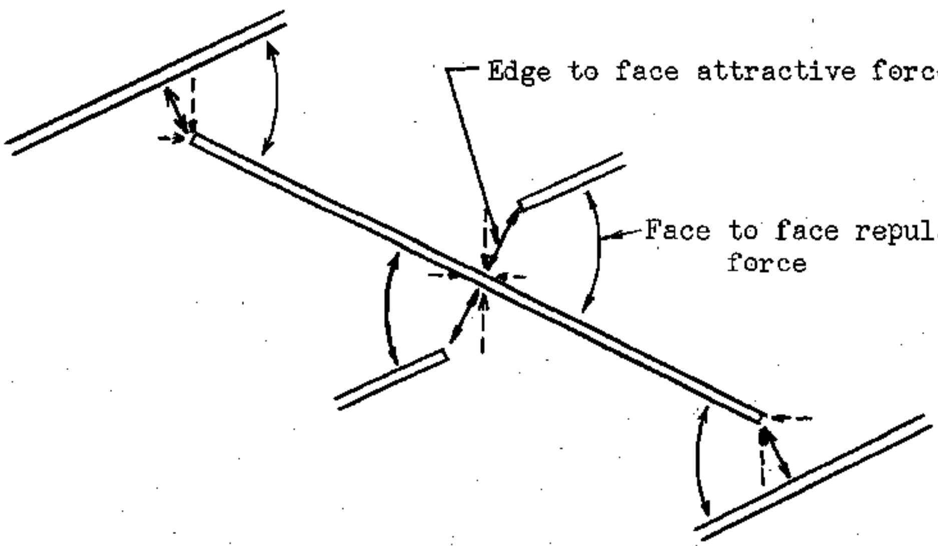
The explanation of these results is further complicated by the large difference in vertical and lateral deformations. For example in Figure 17 the ratio of lateral to vertical strain at the termination of the test was 2.05 corresponding to a ratio of pressures of 1.90. These values are not consistent with usual stress-strain relations.

The only thing in evidence to which this phenomena may be attributed is the anisotorpy of the specimens. Figure 18a shows an idealized two dimensional picture of particles oriented at a 2:1 slope with the horizontal. The system as pictured is not stable; to achieve stability it is necessary that face to face repulsive forces are acting. The idealized force system for an individual particle is shown in Figure 18b.

Assuming that on swelling, the edges move normal to the face of



a. Unit Volume of Soil



b. Force System on Particle

Figure 18. Idealized Particle Arrangement.

the adjacent particle, the resulting vertical unit swell will be four times the lateral unit swell. The reverse will occur for shrinking. With the system restrained so that there is no volume change and, assuming no rotation of particles, swelling would result in a vertical pressure equal to the lateral pressure.

If lateral deformation is allowed but not vertical, the particles move outward, and the ratio of vertical to lateral force at the end of the particle remains constant; and, therefore, the vertical and lateral pressures remain equal. There is a limit to this process depending on the effect of swell on the face to face repulsive forces which are necessary for the system to remain stable.

A system having a particle slope of 1:1 with a horizontal plane would have equal swell in each direction. To obtain a vertical to lateral swell or shrinkage ratio of two, it is only necessary that the particles be oriented only slightly toward the horizontal plane.

With the conditions cited, there would be a tendency for the particles to rotate toward the horizontal. The rotation along with the continued separation of the particles could result in the continued increase in lateral pressure and deformation. As the particles separate the rotational face to face force will decrease making it necessary for the lateral force to increase faster than the vertical. In the system there are twice as many lateral forces as vertical; therefore, the lateral pressure will increase at a faster rate.

It is not meant to be implied in the preceding discussion that the translation and rotation movements in the system are independent and that the rotation occurs after the translation has be completed. For the system to work, it is necessary that the rotation movement be a long time affair, but both movements may start together.

This mechanistic explanation of the data obtained in these tests is presented only as a possible idealized system which could produce such data. The system is too oversimplified and the data insufficient for the above explanation to be taken as anything other than a postulation as to the swell mechanism.

Results of the Double Oedometer Tests

Three double oedometer tests were conducted, each set having a different initial moisture content. The results of these tests are shown in Figures 19, 20 and 21. According to Jennings and Knight (18) the compression curves for a test set should be adjusted so that the virgin portions of the curves coincide. The vertical difference between the adjusted curves is the heave expected if the moisture condition goes from the initial to a flooded condition. It is obvious from the curves obtained in these tests that the adjustment of the curves is questionable. The virgin portions of the two curves of each set are not parallel; therefore, the curves can not be adjusted to coincide. The specimens were loaded to at least 32 kips per square foot which should be sufficient to obtain a virgin curve. It would appear that for highly desiccated or preconsolidated soils, the test must be carried to load magnitudes which are greater than can be obtained in most laboratories.

It is interesting to note that in each test the flooded specimen consolidated at a faster load rate, therefore, the two curves cross. This suggests that due to the release of the negative pore pressure and

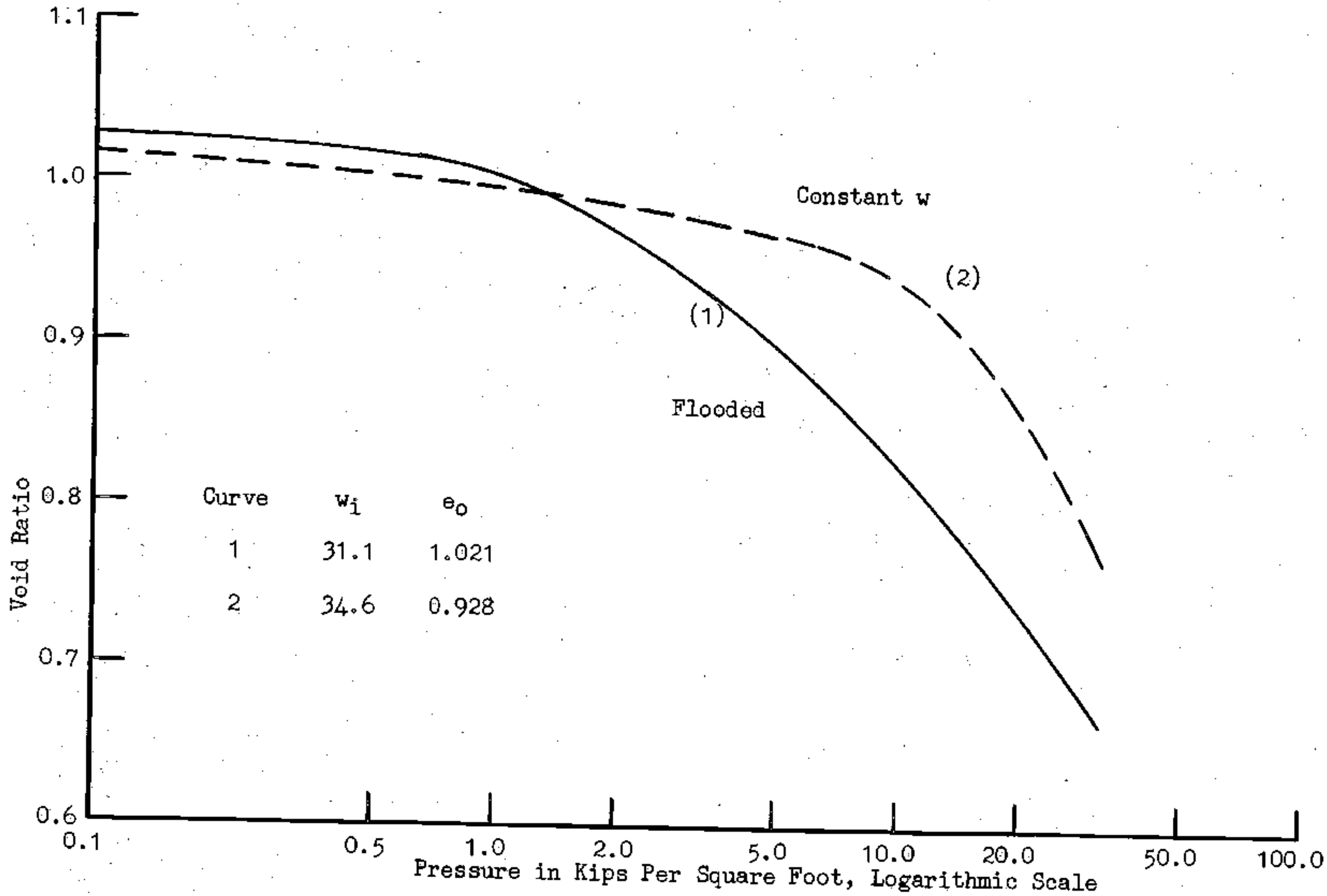


Figure 19. Void Ratio vs Pressure

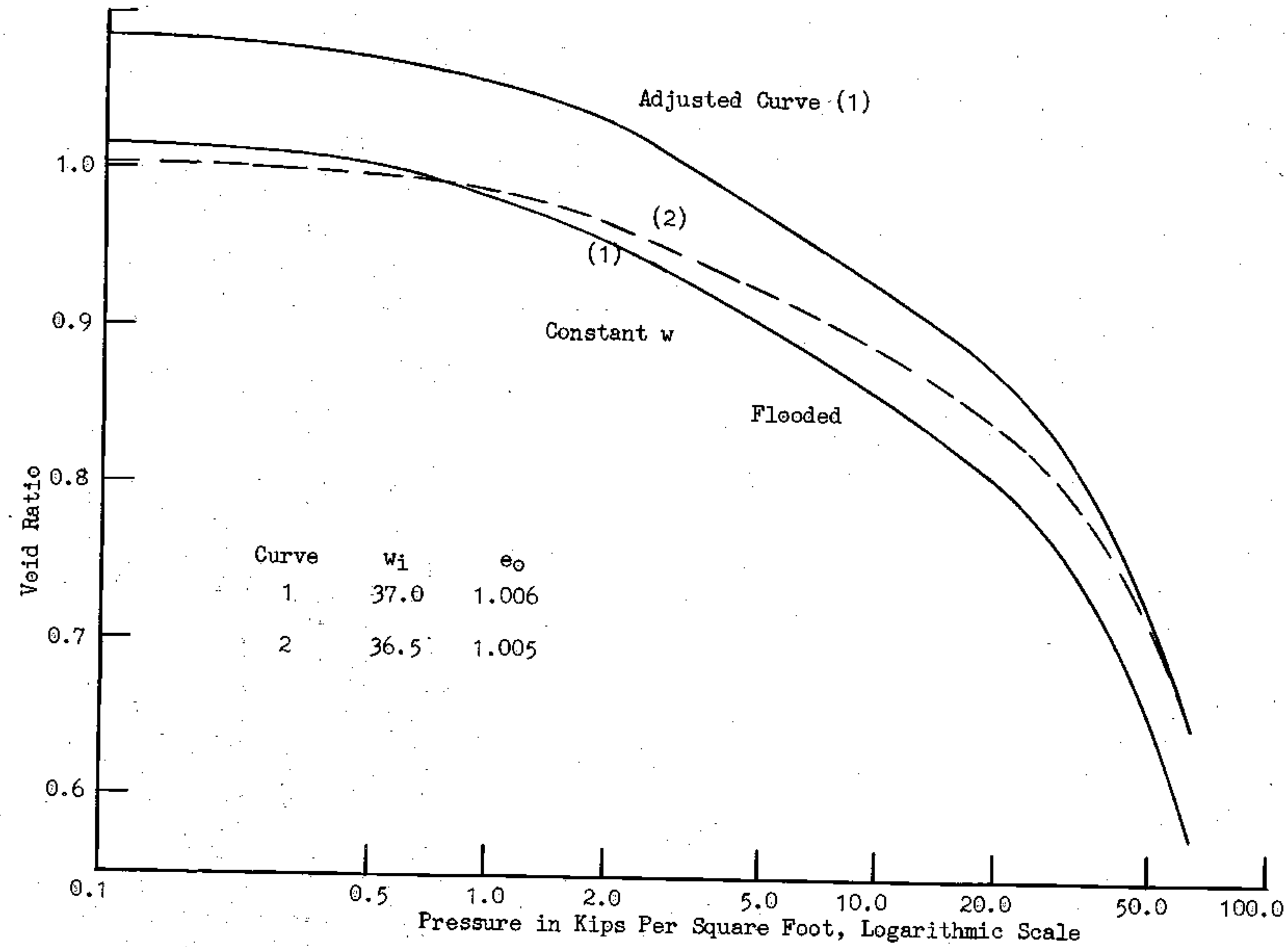


Figure 20. Void Ratio vs Pressure

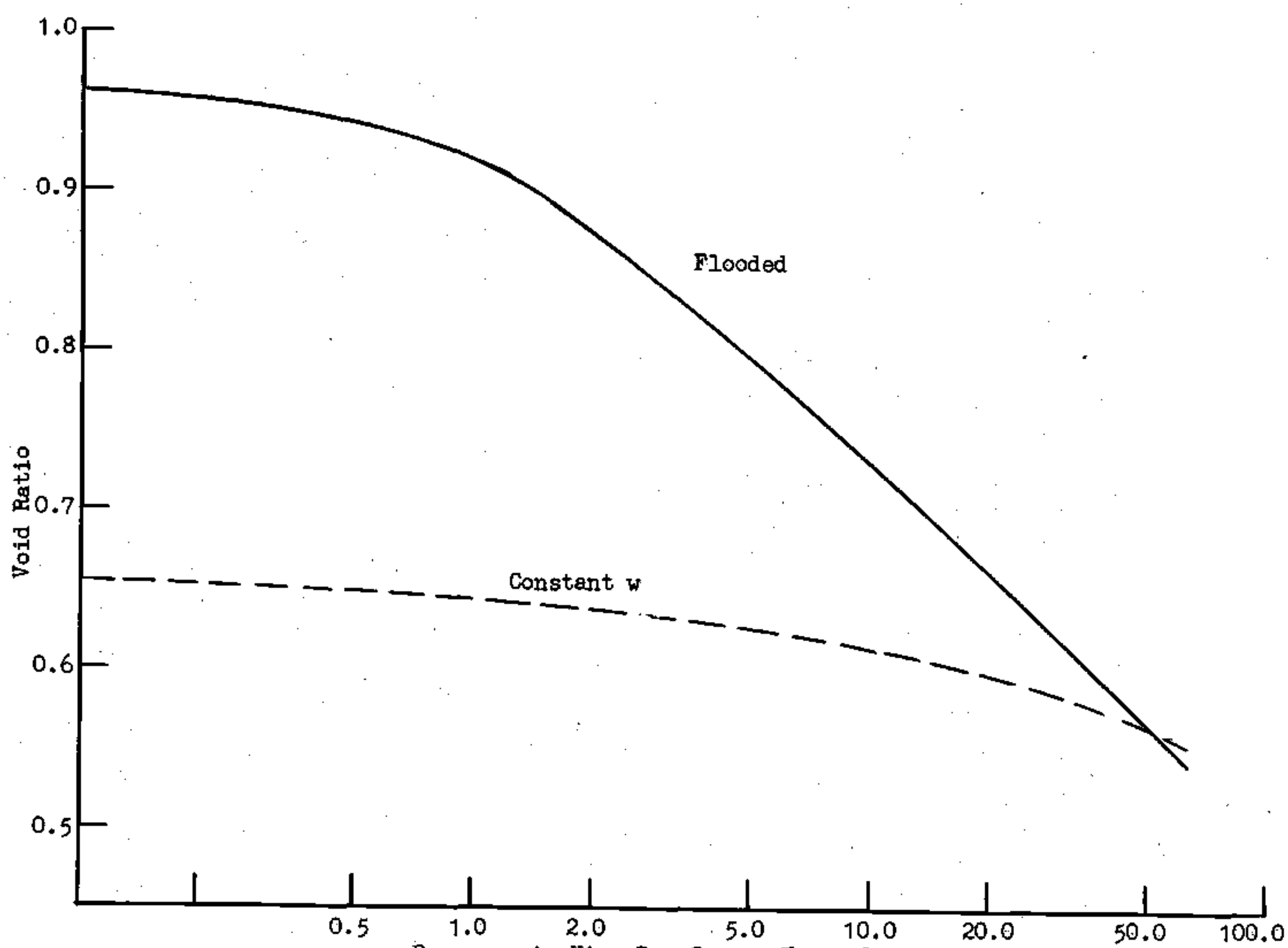


Figure 21. Void Ratio vs Pressure

subsequent swelling, the compression curves of the two specimens do not have the same shape. This may be attributed to the possibility that the clay minerals loaded with the negative pore stress are less likely to orientate themselves normal to the axis of loading than those not loaded. There is no test data to substantiate this possibility.

The specimens used for the test shown in Figure 20 were about at the moisture content at which swelling will not occur and the heave of the flooded specimen was slight. Adjustment of the curves indicates that at the initial moisture content, a pressure of about 3500 psf is required to prevent swelling. This, based on the other data obtained on swell-moisture relation of the same soil, is obviously not correct.

<u>The Relation of Cation Exchange Capacity to Several Index Properties</u>

Cation exchange capacity tests were conducted on several selected specimens representing a wide range in index property values. These tests were made to obtain some idea as to the feasibility of such determinations as an indicator of the expansive characteristics of the soil. The data obtained are not extensive enough to make a thorough analysis of the relation but are intended only as a guide. Each determination represents the average of either two or three determinations on similar samples.

The data are shown in Figures 22, 23 and 24. With the exception of one point, there is a good correlation between the exchange capacity and the liquid limit, plastic limit and per cent colloids. Unfortunately, the values obtained for the cation exchange capacity depend largely on the method used and the manner in which the tests were

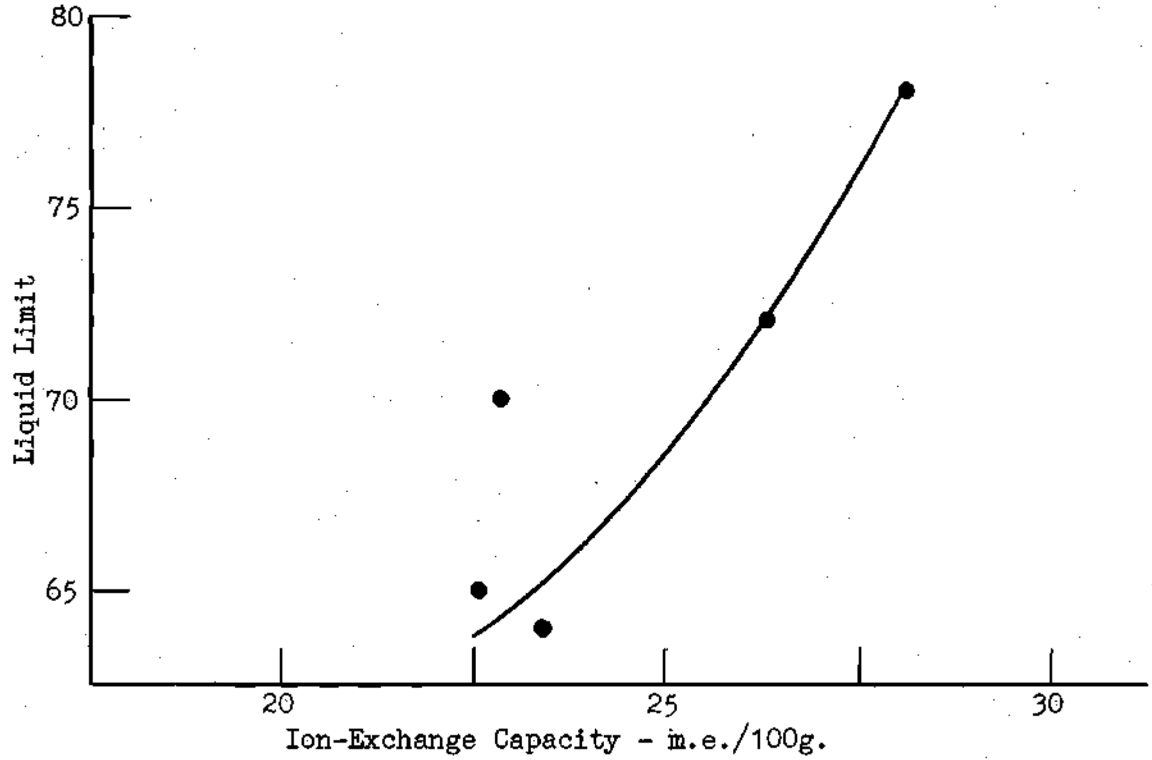


Figure 22. Ion-Exchange Capacity vs Liquid Limit

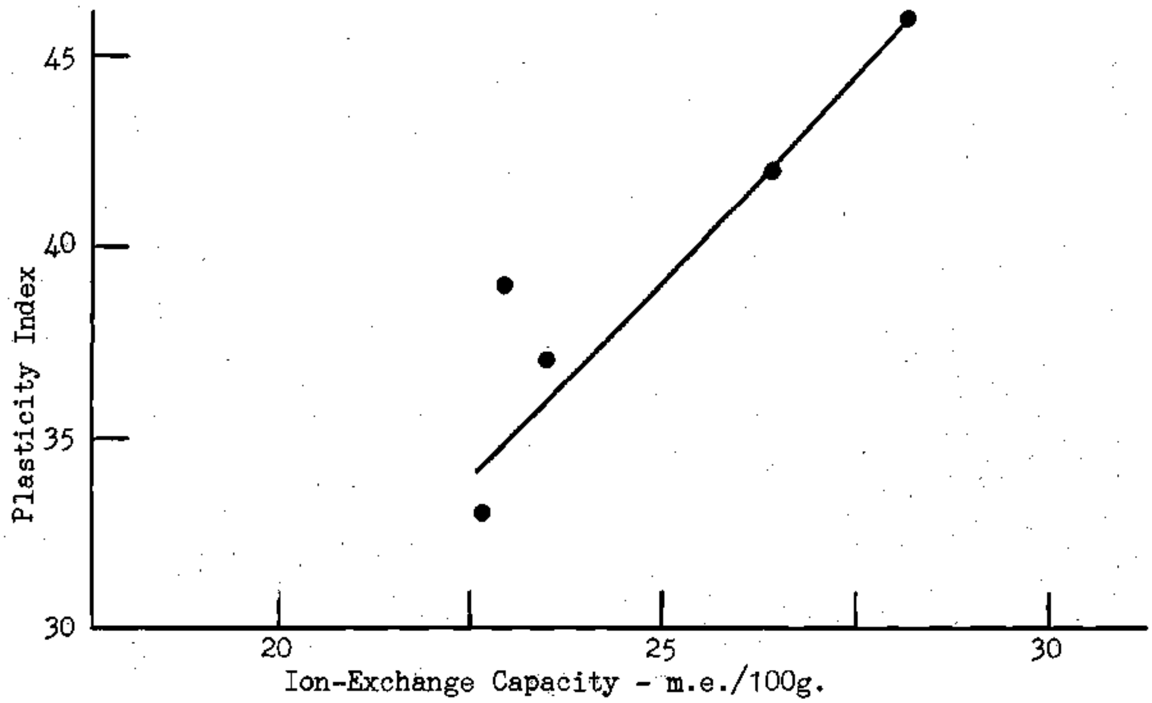


Figure 23. Ion-Exchange Capacity vs Plasticity Index.

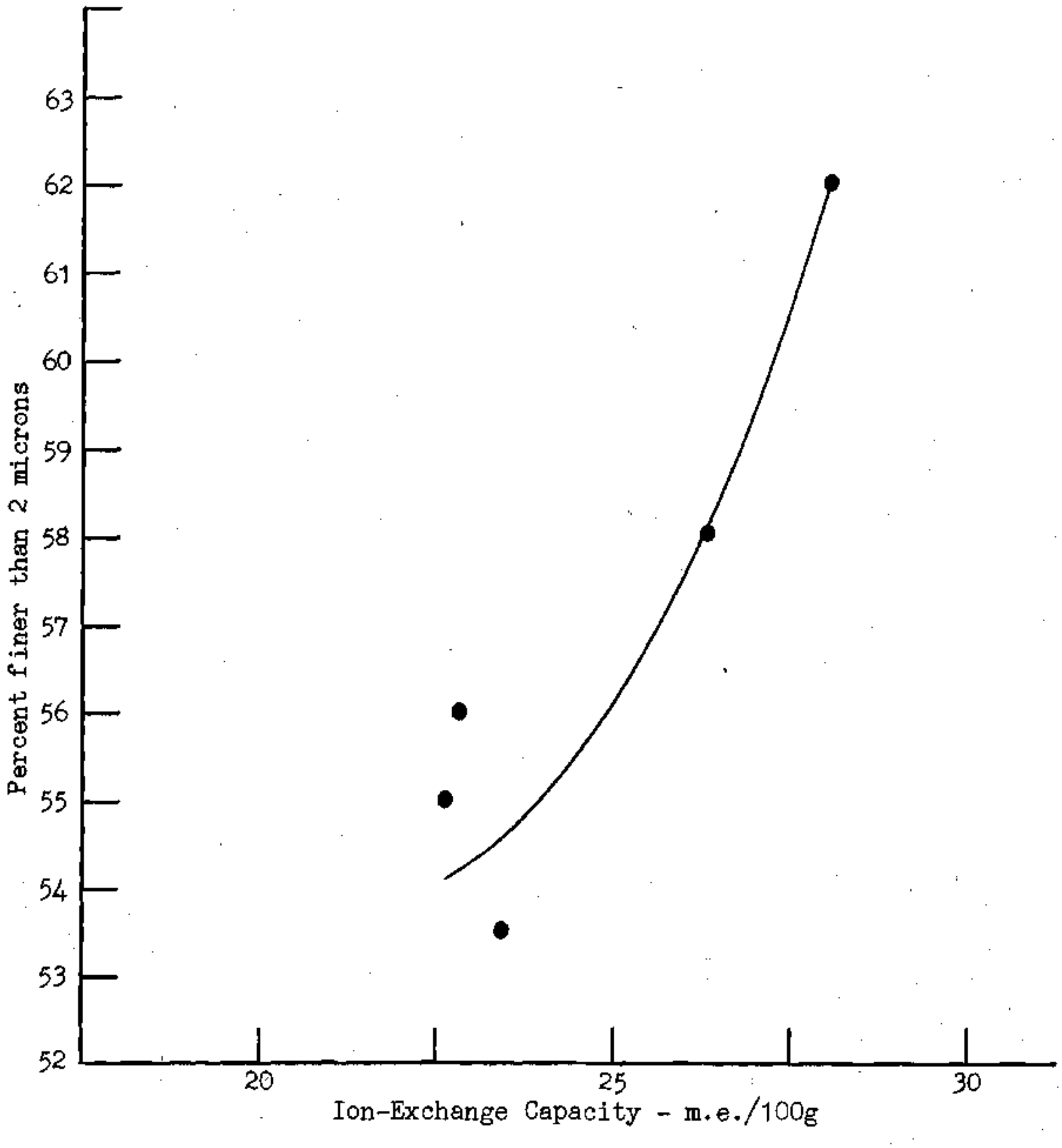


Figure 24. Ion-Exchange Capacity vs Percent Finer Than 2 Microns.

conducted making it difficult to correlate the data with published data.

The cation exchange capacity of a pure clay is indicative of the surface charge on the clay mineral which is the fundamental factor in determining the Atterburg Limits of a soil. By expressing the exchange capacity in terms of the total soil constituents of a natural soil, the exchange capacity becomes indicative of the per cent colloids as well as the liquid limit and plasticity index. This value along with some indication of the nature of the adsorbed ions should offer an excellent index as to the expansive characteristics. Finding an easy, consistent method of determining the exchange capacity and type of adsorbed ions present is necessary before extensive use of these relations can be made.

The Effect of Calcium and Sodium Chloride on Swelling

The relative swell of undisturbed specimens immersed in several concentrations of calcium chloride solutions is shown on Figure 25. Each curve represents a set of specimens taken from one sample. The swell for those specimens immersed in distilled water was taken as 100 per cent.

The results of these tests indicate that as the concentration of $CaCl_2$ increases, the relative swell of the clay is decreased. There is some indication that as the concentration of $CaCl_2$ approaches 5 molar, the swell becomes constant.

Based on cation exchange capacities determined on similar specimens and the assumption that all of the adsorbed ions are calcium, rough computations show that the ion concentration in the adsorbed layer is

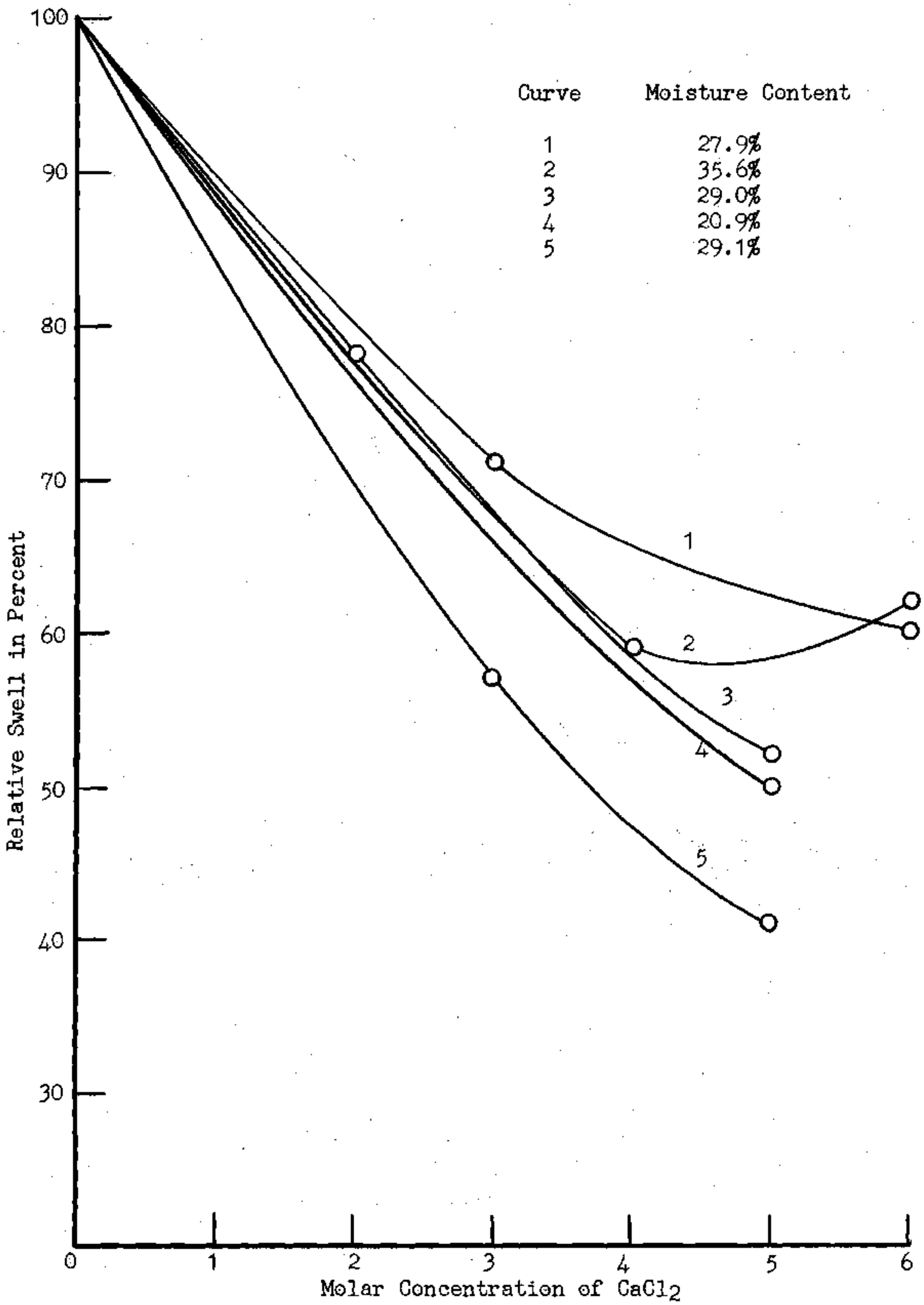


Figure 25. Relative Swell for Specimens Immersed in $CaCl_2$ Solutions.

less than 2.5 molar. There is no indication from the data that there is any sharp change in the swelling as the concentration of ions in the pore water becomes equal to and surpasses the concentration in the adsorbed water layer.

Initially there is osmotic* swelling for each concentration until the calcium chloride ions diffuse throughout the pore system. If the osmotic swelling is well advanced before the ion concentration reaches a maximum, there should be a decrease in volume when the pore water concentration becomes equal to or greater than the adsorbed water concentration, thereby destroying the osmotic system. Evidence of this may be seen on the time-swell curves of Figure 26. The data shown here indicate a continuing increase, although very slight, for the distilled water and a loss of volume for the calcium chloride solution after a maximum value has been reached.

The decrease in swell for concentrations less than or equal to the ion concentration in the adsorbed layer may be attributed to loss of osmotic pressure; for higher concentrations the decrease in swelling becomes more complex.

In a true osmotic system an increase in concentration on one side of the membrane from less than to greater than the concentration on the other side of the system results in a reversal of the osmotic pressure. The osmotic system in soils is one without membrane where

*The term osmotic is in the same sense as in other soil mechanics papers, i.e., the condition of different ion concentrations between adsorbed water and pore water.

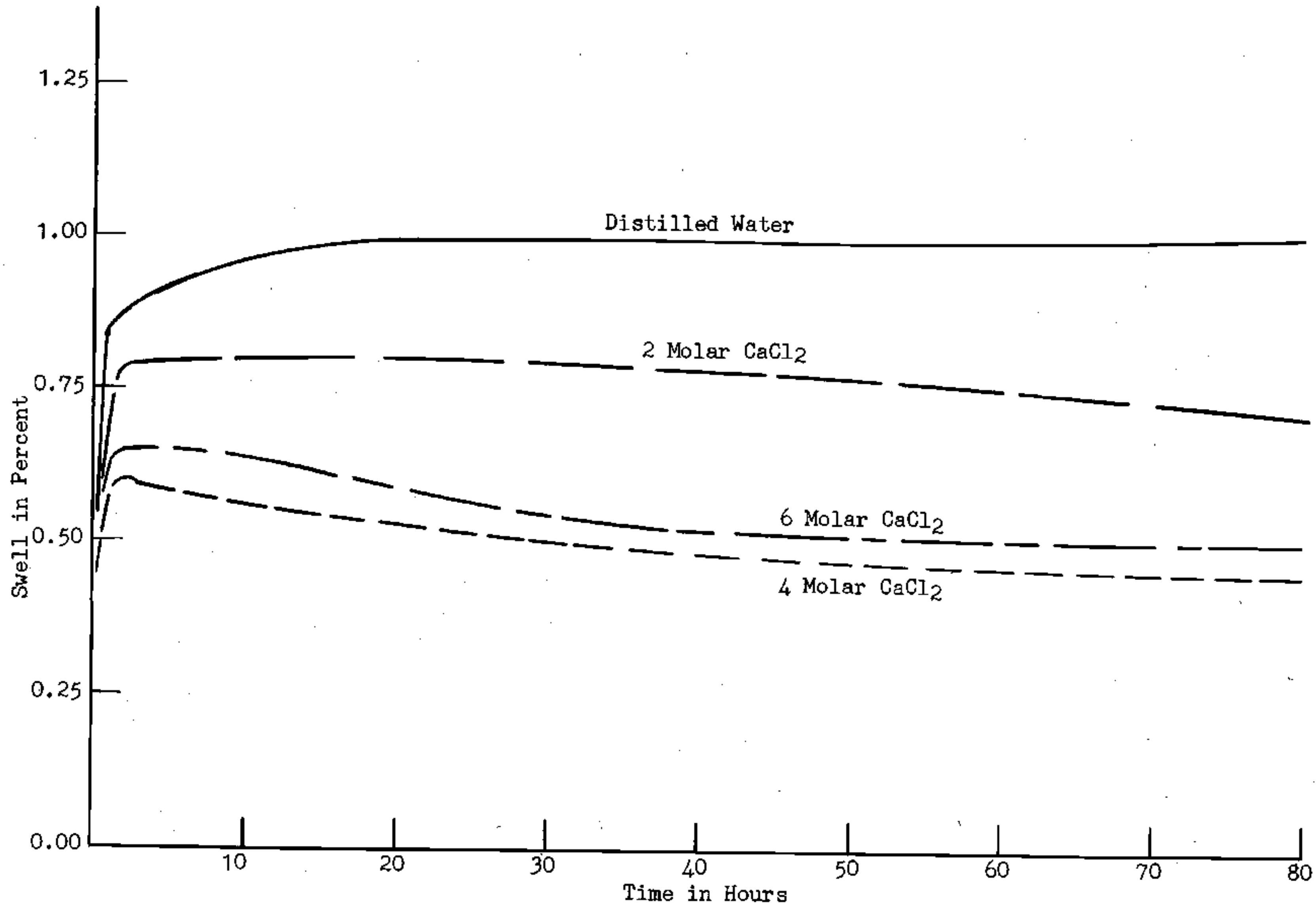


Figure 26. Typical Time-Heave Curve for Specimens Immersed in $CaCl_2$.

the forces preventing the adsorbed ions from diffusing into the pore water are not capable of preventing ions from being forced into the adsorbed layer. This suggests that once the ion concentration in the pores becomes equal to the concentration in the adsorbed water, any further increase in ion concentration of the pore water results in an equal increase in the adsorbed water. There may be a loss of osmotic pressure but no reversal; therefore, the continued decrease in swelling for high concentrations must be attributed to the presence of the salt solution on other phenomena which contribute to swelling.

One possibility which may be considered is the shielding effect of the increased concentration of ions in the adsorbed layer resulting in a reduction of electrostatic repulsion between clay particles.

Although there is no evidence available to correlate the activity coefficient of water molecules in solution with the swelling of clays, the fact that this coefficient does vary with the $CaCl_2$ concentration introduces a question unanswerable at this time.

No effort has been made to explain the role of the chloride anions in these results. This subject has received very little attention in past research resulting in almost no knowledge of the aspect.

Figure 27 shows the relative swelling for specimens immersed in several sodium chloride solutions. The increase in swelling for the two molar solution suggests an immediate exchange of calcium ions for the sodium ions, verifying the assumption that the clay minerals of this soil are primarily a Ca-clay.

It is generally considered more difficult to exchange calcium ions than sodium ions. Realizing that there are many factors involved

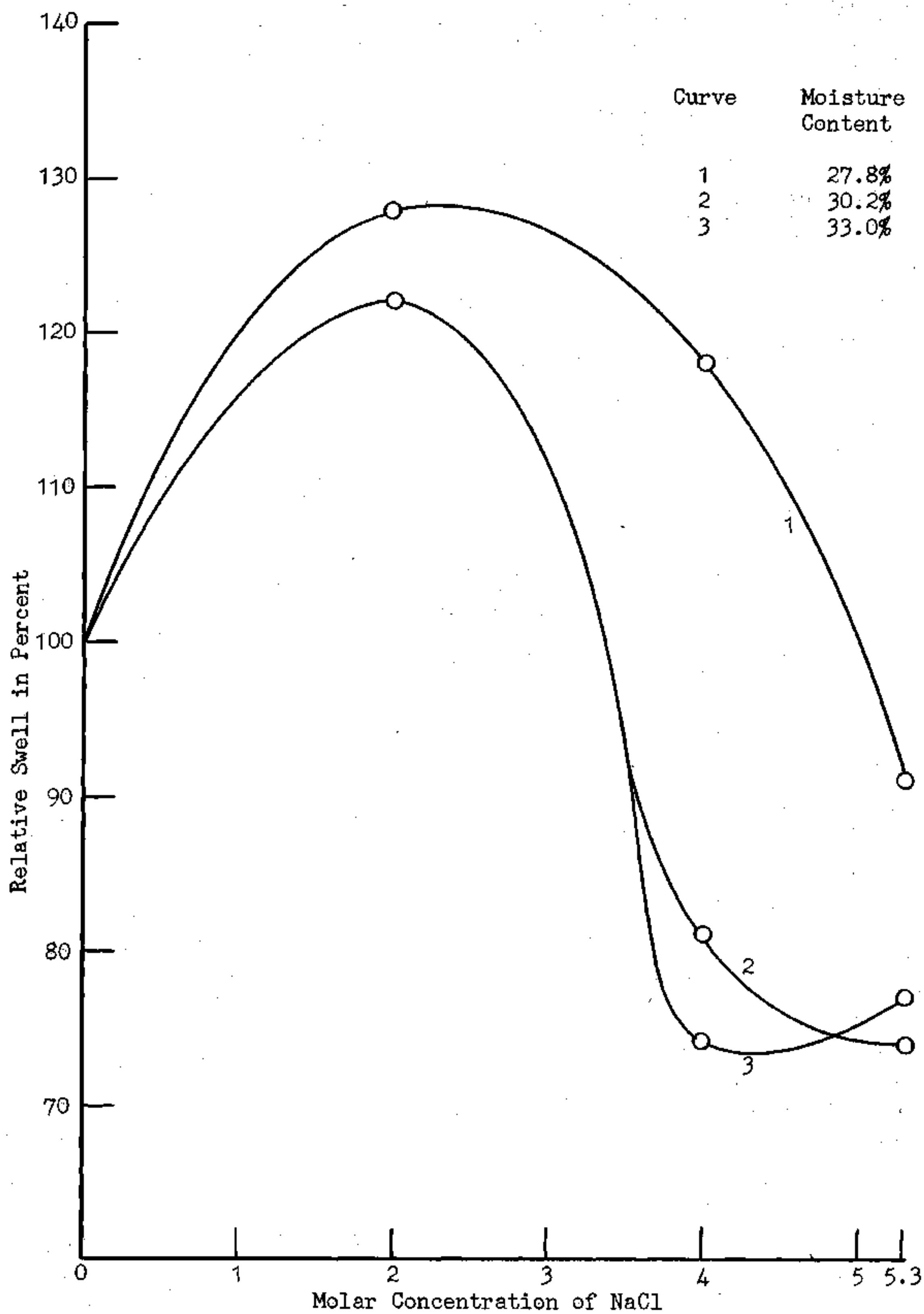


Figure 27. Relative Swell for Specimens Immersed in NaCl Solution.

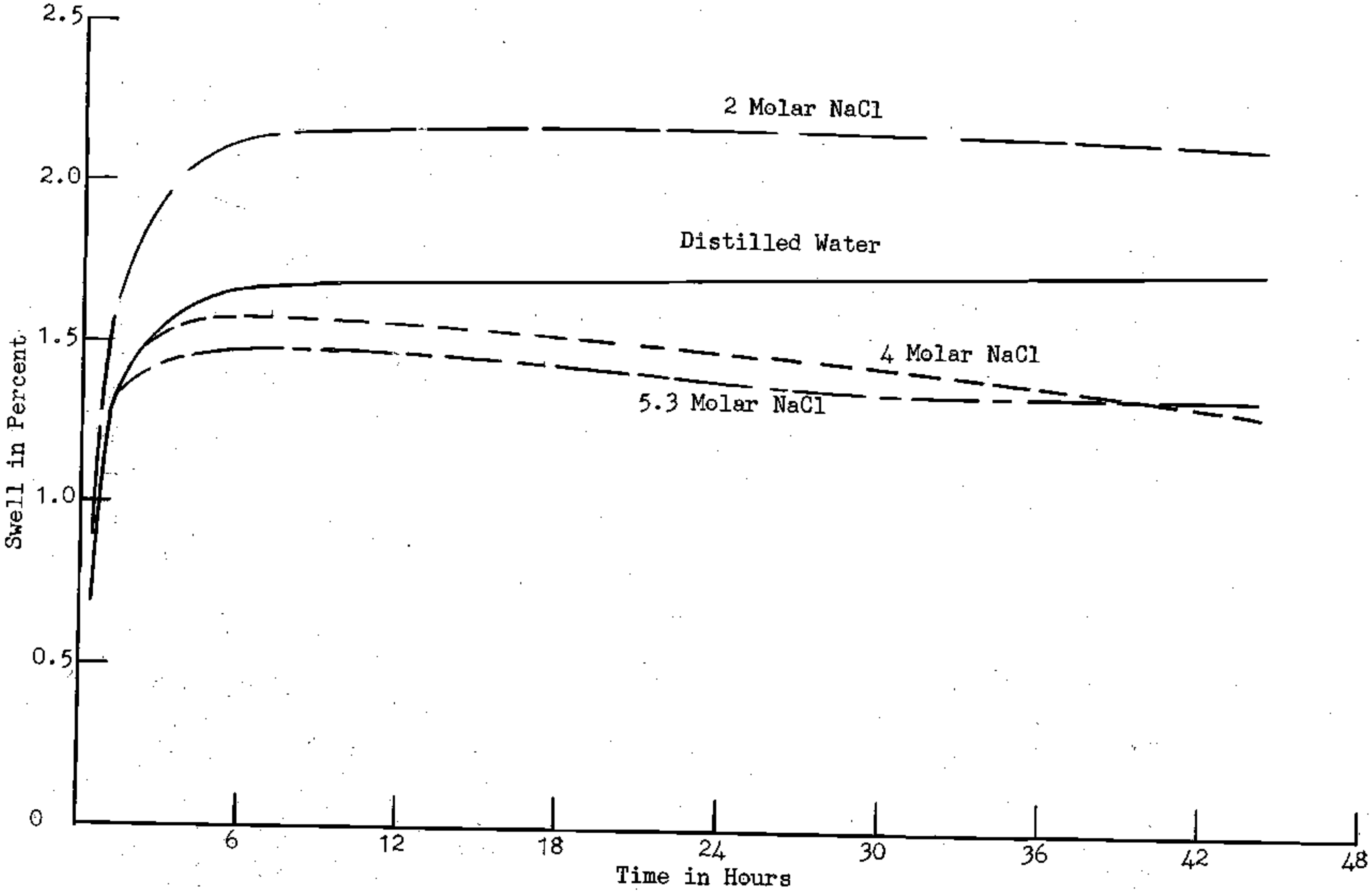


Figure 28. Typical Time-Heave Curve for Specimens Immersed in NaCl.

in ion exchange, the occurrence of the exchange for the two molar solution indicates that this concentration is greater than the calcium concentration in the adsorbed layer.

Once the exchange process has reached an equilibrium, an increase in the $NaCl_2$ concentration in the pore water would be similar to the conditions which occurred in the initial $CaCl_2$ test. This explains the reduction in swelling from the peak value.

The variation of Curve 1 from Curves 2 and 3 in Figure 27 is probably due to a high ion exchange capacity for this particular sample.

Taking the value of 2.3 molar as the probable maximum concentration of the adsorbed ions, the data in Figure 25 indicate that osmotic swelling for the Yazoo Clay is less than one-fourth of the total for the moisture contents tested. Although the range in moisture content is not extensive, there is no indication of any correlation between initial moisture content and reduction of swelling.

If we attribute the remaining reduction in swell to the shielding effect, the data indicate that electrostatic repulsion may produce more than one-fourth of the swelling. The remainder of the swelling may be attributed to the other phenomena, such as hydration of adsorbed ions.

The above breakdown on the causes of swelling is not based on extensive data and is presented to show the complexity of the swelling problem and to suggest possible avenues for future research. There is also some indication that any theoretical approach to the problem of computing swell or swell pressures can not be based on one phenomenon, at least for the Yazoo Clay.

X-Ray Diffraction Test Results

The X-ray diffraction tests were undertaken to determine if the X-ray diffraction trace obtained from powdered, untreated specimens would reveal any clue that may be used as an indicator of the expansive property of a soil. It is, of course, possible, by using the various techniques which have been developed, to determine what minerals are present, from which the expansive nature of the soil may be estimated. Such techniques are too time consuming and costly for the desired purpose.

Figure 29 shows the results of tests on four specimens as outlined in the testing program. The original machine traces possessed a high background indicating that the specimens were rich in iron. The traces as shown (excepting No. 2) are dominated by the 3.04 line which is the result of a high calcium carbonate content.

The combination of the iron and the calcium carbonate make it difficult to pick up all of the lines necessary to make an analysis. Despite this difficulty there are several interesting things which should be noted. The height of the 3.04 line on the original trace of Specimen 1 was approximately four times the height of the same line on the trace for Specimen 2. This suggests that the calcium carbonate content of the weathered clay is much higher than for the unweathered. This may account for the difference in plasticity index which is about 60 for the unweathered and 40 for the weathered clays tested. The unweathered clay contains many pieces of thin shell which are not apparent in the weathered clay. It is possible that these concentrations of calcium carbonate decompose in the weathering process and are diffused throughout the soil.

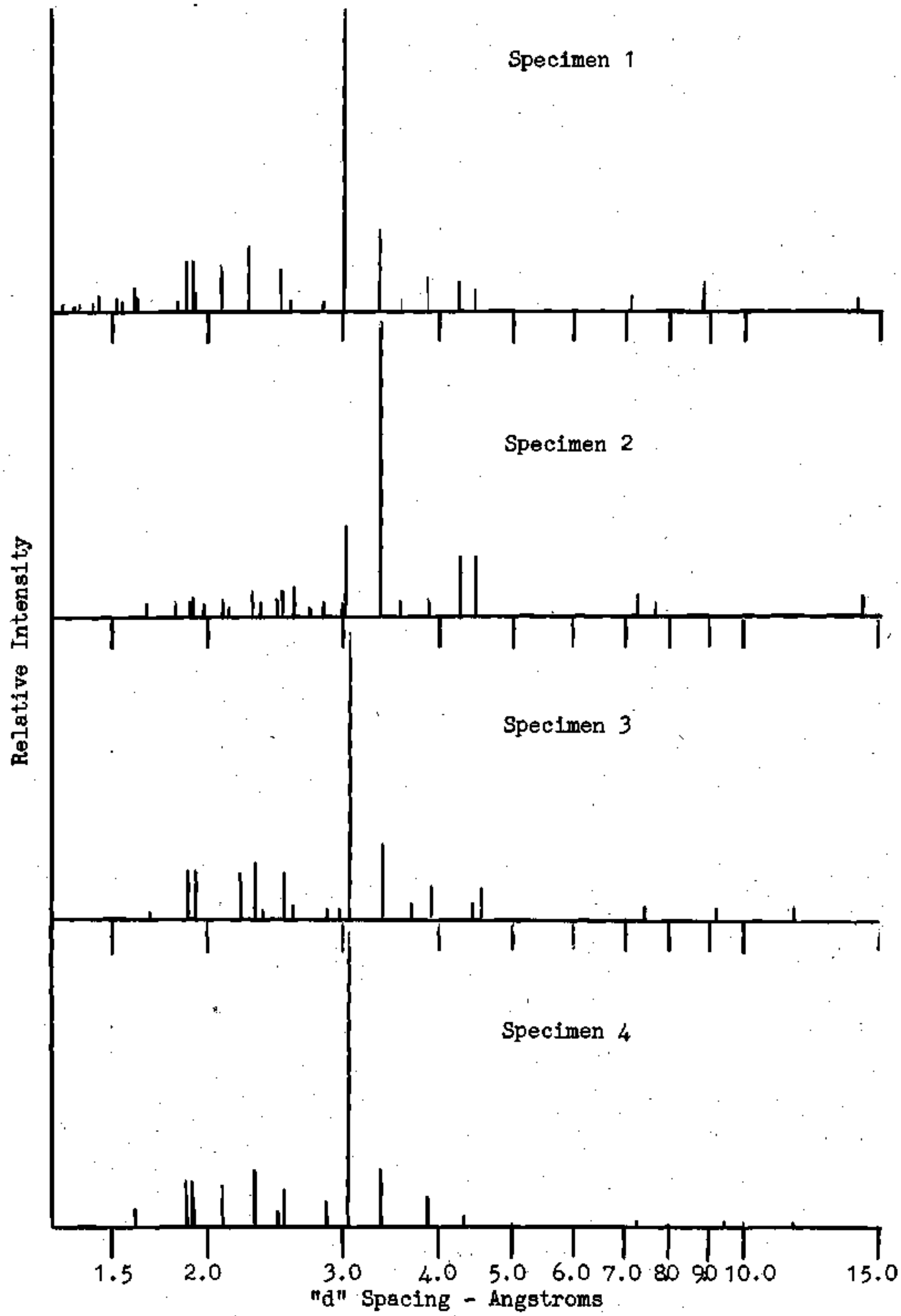


Figure 29. X-Ray Diffraction Data for Yazoo Clay.

The X-ray specimens of unweathered clay contained no visible traces of shells which may account for the lower carbonate content.

Considering the manner in which Specimens 3 and 4 were obtained, No. 3 should be high in clay and No. 4 very low. The only indication of this condition is the difference in relative intensities of the high spacings. Generally, expansive clays will have lines with d-spacings above eight or nine. By observing the relative intensity at these lines, it may be possible to judge the expansive nature of a soil.

Figure 30 represents traces made on undisturbed slices having different orientation. The only difference in these traces which suggests that there is preferred orientation is the difference in intensity and width of the 19.0 d-spacing line. It is quite possible that the cutting process used in preparing the specimens produces disturbance to a depth greater than that penetrated by the X-rays. If this occurs, the traces are more indicative of difference in disturbance than difference in orientation. Assuming this is not the case, the slight orientation is in line with that needed in the idealized arrangement previously discussed to produce the difference in vertical and lateral shrinkage.

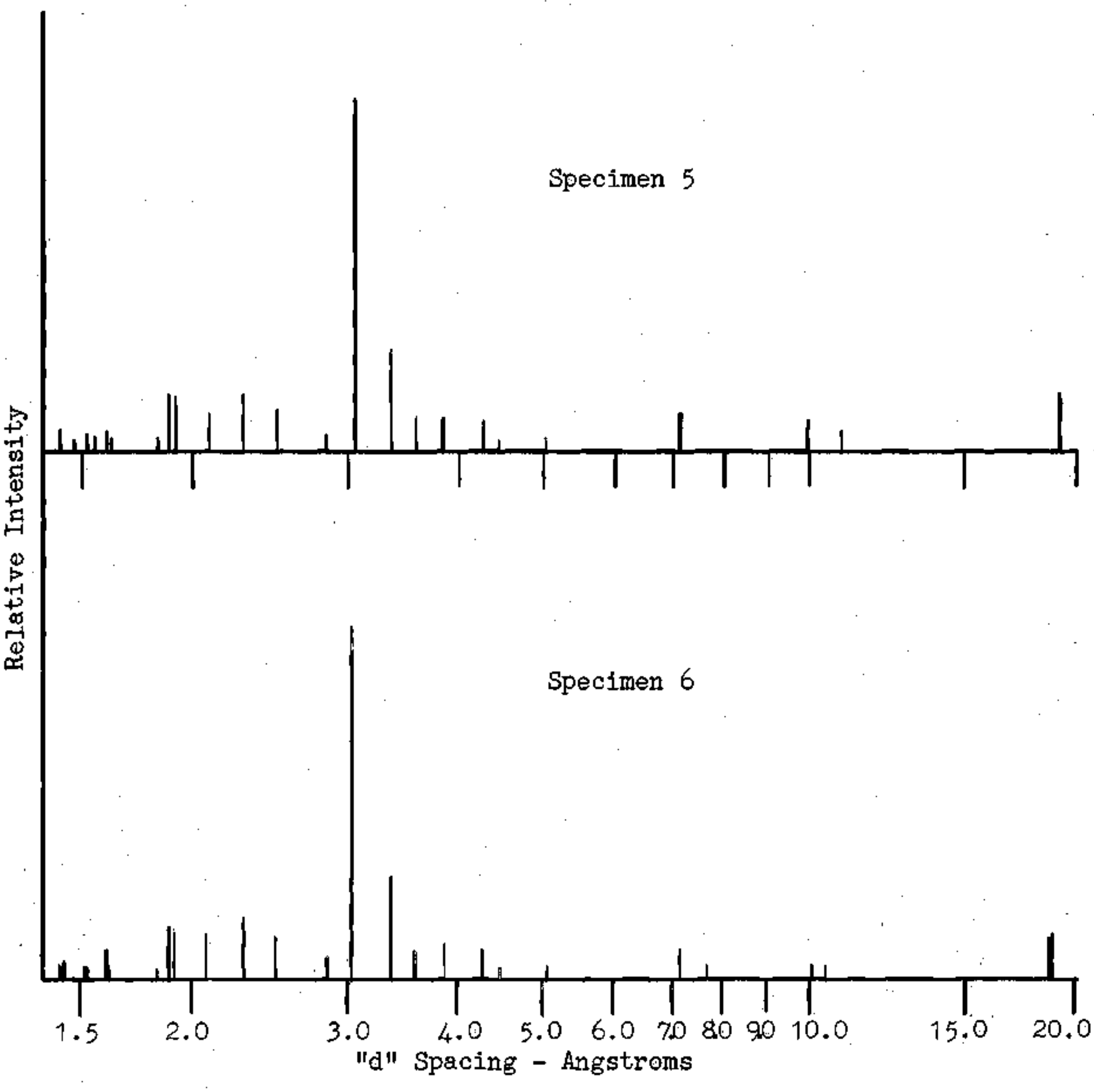


Figure 30. X-Ray Diffraction Data for Yazoo Clay.

CHAPTER VIII

CONCLUSIONS

(1) For near zero volume change, the swell pressure developed when the Yazoo Clay is allowed to imbibe water freely is dependent on the initial moisture content. As the initial moisture content decreases, the pressure developed increases.

(2) The swell pressure for a given moisture content can vary considerably for samples secured within a few feet due to the erratic changes in soil as noted by the plasticity index.

(3) For all practical purposes, the Yazoo Clay will not swell when moisture content reaches about 1.2 times the plastic limit.

(4) Under some but presently undefined conditions, the Yazoo Clay will continue to swell for many days at a slow rate which, with enough time, can become significant.

(5) The pressure required to consolidate an undisturbed natural clay to a given void ratio is greater than the pressure developed if the specimen is desiccated to the same void ratio and then inundated and prevented from swelling. The compression curve obtained from the consolidation test is not indicative of the swell pressure-void ratio relation of a clay soil.

(6) Vertical and lateral pressures produced by primary swelling are approximately equal for small deformations as long as the ratio of vertical to lateral deformation remains in the approximate range of 0.5 to 2.

(7) Swelling which results from a difference in cation concentration in the adsorbed and pore water accounts for only a small portion of the total swell of the samples tested.

(8) Swelling of a natural clay can be reduced by the introduction of salts into the pore water, but it is necessary that it contain the same cation as adsorbed on the clay.

(9) The Yazoo Clay, at least in the weathered portion, has a preferred orientation of clay particles which results in greater swelling or shrinking normal to the bedding plane than parallel to it.

(10) The shrinkage characteristics of undisturbed clay specimens should be valuable as an indication of the anisotropic nature of the soil as well as its expansive properties.

(11) The determination of the shrinkage limit using remolded samples is of little value when studying the shrinkage characteristics of the Yazoo Clay.

(12) The undisturbed clay has a natural shrinkage limit above which the void ratio increases directly with the moisture content. The degree of saturation above the natural shrinkage limit varies between 90 and 100 per cent.

(13) The double oedometer test of Jennings and Knight (18) is not reliable as a method of predicting heave because of an inherent difference in the shape of the compression curves for specimens tested under flooded conditions and those tested at constant moisture content.

(14) When it is possible to find an easy method which can be standardized for determining the ion exchange capacity and type of adsorbed ion, this property may offer an excellent indication of the

swell potential of a natural clay soil.

(15) At present the best indication of the swell potential of a soil is the plasticity index and the natural moisture content as compared to the plastic limit.

(16) X-ray diffraction methods are not sufficiently available and are too expensive to be presently used as a method predicting swell potential.

Implications on Field Practice

The knowledge obtained in this investigation has several implications in engineering practice:

(1) For the purpose of obtaining possible limits in magnitude of the swell-shrinkage characteristics of an expansive soil, the change in void ratio between the natural shrinkage limit and the void ratio corresponding to 1.2 times the plastic limit can be used.

(2) If the moisture content of the soil mass is approximately 1.2 times the plastic limit at the time of construction, swell is unlikely. A simple foundation system utilizing bored piles, grade beams and slab poured directly on the ground can be used because only shrinkage of the soil is likely to occur.

(3) The practice of allowing undisturbed clay specimen to swell in a consolidometer, then determining the load required to consolidate it back to the original void ratio and using this value as the pressure required to prevent swelling is questionable. The values obtained in this manner are too high.

Swelling tests should be continued for sufficient time (two weeks) so that secondary swelling, if present, can be noted.

(4) The variation in swelling and shrinking in directions perpendicular to and parallel to the bedding plane of the soil should be considered in relation to the structure and the dip of the stratification or bedding of the clay.

CHAPTER IX

RECOMMENDATIONS FOR FURTHER STUDY

In discussing the objectives of this study, it was implied that the research was of an exploratory nature and that it would be impossible to explore fully each facet of the overall problem. The data obtained is insufficient and should be supplemented with additional research. Special consideration should be given to the following.

(1) Long time testing under conditions of strict environmental control is needed in studying the time effect on both swelling and shrinking of soils. Special emphasis should be placed on the study of relative vertical and lateral pressures under no volume change conditions and under various ratios of vertical and lateral strain.

(2) The use of shrinkage measurements as an indicator of anisotropy of soils should be considered. A comprehensive testing program is necessary to study the effect of particle orientation in desiccated soils on the various engineering properties.

(3) Further study is needed in the problem of comparing void ratio-swell pressure and consolidation test data. The consolidation test is the most widely used means of determining loads to prevent swelling. There is evidence that these values are excessive.

(4) There is need for much work on the problem of simplifying the procedures for identifying adsorbed ions and determining the cation capacity of soils for the expressed purpose of arriving at a method

which may be conducted in the average commercial soils laboratory.

(5) An effort should be made by soil mechanics engineers to adapt a system of expressing the expansive properties of a soil so that future experience and research may be better correlated.

(6) Field studies are needed to determine the depth of seasonal moisture changes and the effect change in surface environment has on subsurface moisture conditions.

(7) Studies should be made in the field to determine the feasibility of modifying the properties of expansive soil masses by the use of concentrated salt solutions placed in holes drilled into the soil.

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